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Seismic Safety Evaluation of Existing San Francisco Bridges

Volume I

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Group IIA - Concrete Bridges
- Market Street Viaducts 1 through 6
- Harrison Street at Beale Street
- Bryant Street between Second and Beale Streets
- Mission Street at Alemany Boulevard

Group V - Steel Bridges
- Third Street Bridge over Channel
- Fourth Street Bridge over Channel
- Third Street Bridge over Islais Creek

Prepared for:
City and County of San Francisco
Department of Public Works
Bureau of Engineering
Room 351, City Hall
San Francisco, California 94102

Prepared by:
URS/John A. Blume & Associates, Engineers
130 Jessie Street (at New Montgomery)
San Francisco, California 94105

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SEISMIC SAFETY EVALUATION
OF EXISTING SAN FRANCISCO BRIDGES

for the

CITY AND COUNTY OF SAN FRANCISCO

INTRODUCTION

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under the Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridge up to current seismic safety standards, and if remedial work is required, to recommend a remedial program and estimate the cost.

This report is presented in two volumes:

Volume I: Summary of Findings

Group IIA - Concrete Bridges

- Section IIA-1. Market Street Sidehill Viaduct No. 1
- Section IIA-2. Market Street Sidehill Viaduct No. 2
- Section IIA-3. Market Street Sidehill Viaduct No. 3
- Section IIA-4. Market Street Sidehill Viaduct No. 4
- Section IIA-5. Market Street Sidehill Viaduct Nos. 5 & 6
- Section IIA-6. No section
- Section IIA-7. Harrison Street Bridge over Beale Street
- Section IIA-8. Bryant Street Viaduct between 2nd and Beale Streets
- Section IIA-9. Mission Street Viaduct over Alemany Boulevard

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Group V - Steel Bridges

- Section V-1. Third Street Bridge over Channel
- Section V-2. Fourth Street Bridge over Channel
- Section V-3. Third Street Bridge over Islais Creek

Appendix A: Evaluation Criteria

Appendix B: Caltrans Seismic Response Spectra

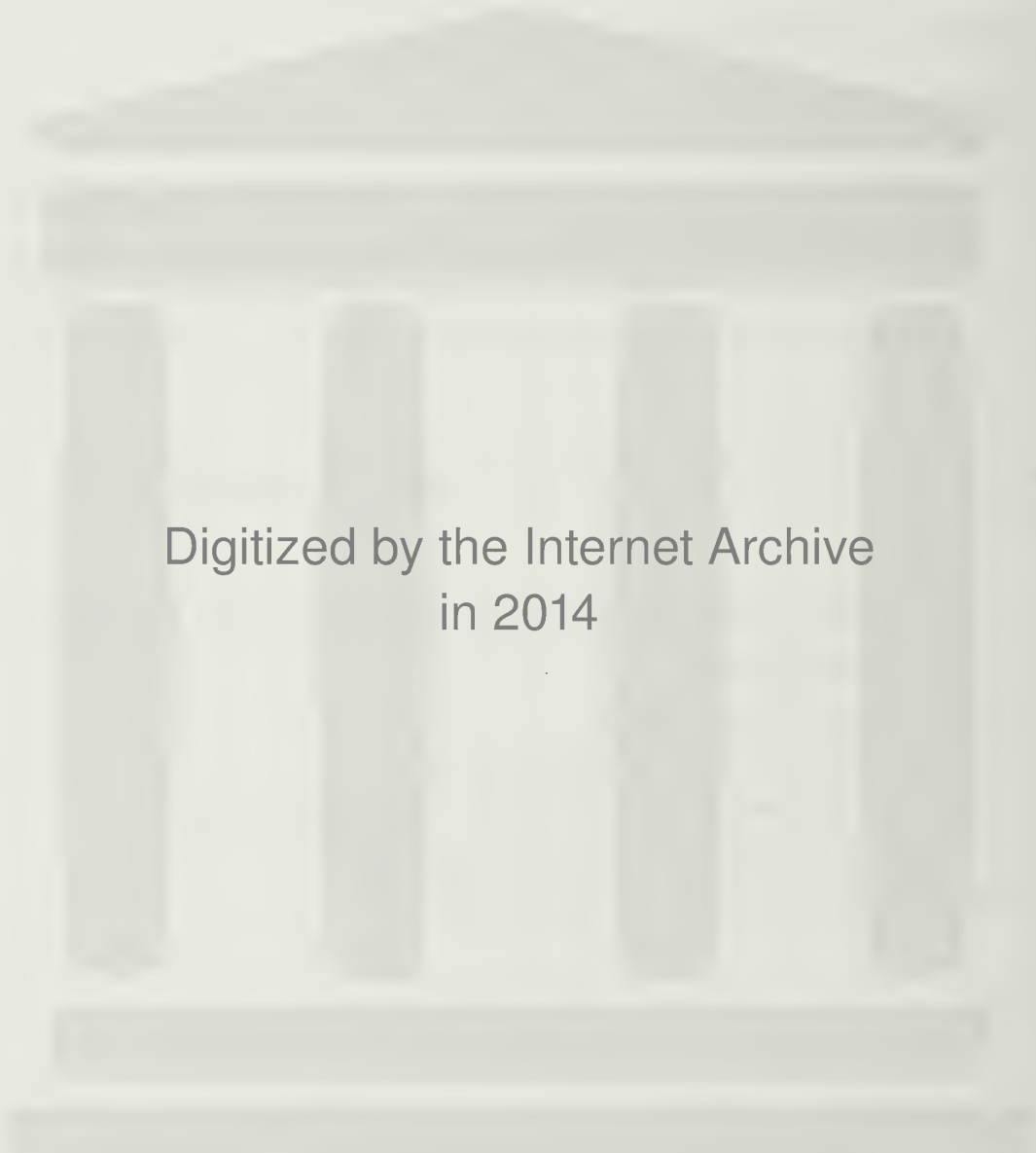
Appendix C: References

Volume II: Allstate Geotechnical Services, "Geotechnical Investigation - Seismic Safety Evaluation of Existing San Francisco Bridges," prepared for URS/John A. Blume and Associates, Engineers, September 1984. It is referred to in the reports as "Reference 3."

Each section "stands alone" as a separate report and includes its own Table of Contents.

Of the twelve bridges investigated in the study, only the following will require seismic strengthening:

- Market Street Viaduct No. 3
- Harrison Street Bridge at Beale Street
- Bryant Street Viaduct between Second and Beale Streets
- Mission Street Viaduct at Alemany Boulevard
- Fourth Street Bridge at Islais Creek



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SUMMARY

Bridge IIA-1:

Market Street Sidehill Viaduct #1

Northbound Market Street	Between Short and Stanton Streets
Length:	370 feet
Number of Spans:	19
Remedial Work Required:	None

Bridge IIA-2:

Market Street Sidehill Viaduct #2

Northbound Market Street	Between Romain and Morgan Alley
Length:	300 feet
Number of Spans:	16
Remedial Work Required:	None

Bridge IIA-3:

Market Street Sidehill Viaduct #3

Northbound Market Street	Near 23rd Street
Length:	560 feet
Number of spans:	29
Remedial Work Required:	Provide bent cap confinement
	Provide transverse shear lugs
	Provide longitudinal restraining cables
Estimated Cost:	$171,000
Priority:	High

Bridge IIA-4:

Market Street Sidehill Viaduct #4

Southbound Market Street	Near 23rd Street
Length:	300 feet
Number of Spans:	16
Remedial Work Required:	None

Bridge IIA-5 & 6:

Market Street Sidehill Viaducts #5 & #6

#5 - Northbound Market Street at Corbett Avenue		
#6 - Southbound Market Street at Corbett Avenue		
Length:	#5	210 feet
	#6	170 feet
Number of Spans:	#5	11
	#6	9
Remedial Work Required		None

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Bridge IIA-7:

Harrison Street Viaduct Over Beale Street

Length:	180 feet
Width:	66' roadway
Number of Spans:	3
Remedial Work Required:	Add bearing plates in vertical joint
Estimated Cost:	$6,420
Priority:	Low

Bridge IIA-8:

Bryant Street Viaduct Between Second and Beale Streets

Length:	
Width:	
Number of Spans:	
Remedial Work Required:	Install shear walls Provide transverse shear restrains
Estimated Cost:	$233,000
Priority:	Very Low

Bridge IIA-9:

Mission Street Viaduct Over Alemany Boulevard

Length:	179 feet
Width:	58 feet, 6 inches
Number of Spans:	2
Remedial Work Required:	Place shear lugs in abutments
Estimated Cost:	$102,500

Bridge V-1:

Third Street Bridge Over Channel Street Waterway

Type:	Steel single leaf bascule
Span:	142 feet
Width:	80 feet
Remedial Work Required:	None

Bridge V-2:

Fourth Street Bridge Over Channel Street Waterway

Type:	Steel single leaf bascule
Span:	94 feet
Width:	61 feet
Remedial Work Required:	Reinforce trunnion posts
Estimated Cost:	$128,000

Bridge V-3:

Third Street Bridge Over Islais Creek

Type:	Steel double-leaf bascule
Span:	97 feet
Width:	100 feet
Remedial Work Required:	None

TOTAL COST OF RECOMMENDED STRUCTURAL MODIFICATION	$641,000

Related Costs for Additional Work for Drainage and Erosion Control and Soil Stability:

Market Street Viaducts	
Check drainage systems and repair erosion damage	
6 bridges at $3,000	$18,000
Bridge IIA-7: Harrison Street Viaduct Over Beale Street	
Erosion control: Gunite portion of abutment slopes	$3,000
Bridge IIA-8: Bryant Street Viaduct Between 2nd and Beale Streets	
Check and clean weep holes	$3,000
Bridge IIA-9: Mission Street Over Alemany	
Alleviate potential settlement at southwest abutment by grouting	$18,000
Bridges V-1, V-2, and V-3	
Alleviate potential liquefaction by grouting	
3 bridges at $23,000	$69,000
TOTAL COST OF STRUCTURAL MODIFICATIONS AND RELATED SOIL WORK	$767,000

MARKET STREET VIADUCT #1

IIA-1

MARKET STREET SIDEHILL VIADUCT NO. 1

CONTENTS

		page
1.	INTRODUCTION	IIA-1.1
2.	SITE INVESTIGATION	IIA-1.2
3.	BRIDGE DESCRIPTION	IIA-1.4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING BRIDGE	IIA-1.6
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE	IIA-1.7
6.	RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-1.11
7.	SUMMARY AND CONCLUSIONS	IIA-1.12

REFERENCES ..... APPENDIX C

FIGURES:

1.	PLAN AND TYPICAL BENT	IIA-1.13
2.	COMPUTER MODEL	IIA-1.14

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MARKET STREET SIDEHILL VIADUCT NO. 1

1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

2. SITE INVESTIGATION

Results of the site investigation for all bridges is given in Reference 3. A general summary of the reference as it pertains to the Market Street Sidehill Viaduct No. 1 is given in this section.

This viaduct is a cast-in-place concrete vehicular bridge along Market Street between Short and Stanton Streets.

Available geological information indicates that the site is underlain by undivided surficial deposits (Qu). The underlying bedrock is Franciscan Formation of the Central Highlands belt. The depth to the bedrock is estimated to be approximately 20 ft below the sloping ground. An exploratory boring, DH-12, was drilled on Short Street approximately 150 ft from the viaduct to explore the subsurface conditions. The boring was 20 ft deep and the materials encountered consisted of approximately 7 ft medium dense sandy silt (ML) underlain by stiff to hard sandy clay (CL) extending to the depth explored. Standard blow counts of the upper sandy silt material was 14 per foot of penetration and standard blow counts of the underlying sandy clay material ranged from 7 to 49 per foot of penetration. Ground water was not encountered in the boring.

The viaduct is located at approximately 6 miles northeast of the San Andreas fault and 13 miles southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 24 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum II, shown in Appendix B, are recommended for the seismic safety evaluation.

The cohesive nature of the soil underlying the viaduct and the low ground water level suggest that liquefaction is not likely. Plate 10 of Reference 3 shows that the viaduct is located in the area of potential landslide. Our reconnaissance of the site, however, did not indicate any signs of land instability. No slides were identified at the site by available published data. We believe that the risk of landslide during earthquake is not significant. Minor downhill ground movement, however, may be induced by a

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major earthquake. Such movement may result in minor structural cracking. We recommend that periodical maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.

The site is located at a great distance from the areas of potential lateral spreading and tsunamni inundation as shown on Plates 3 and 11 of Reference 3.

3. DESCRIPTION OF BRIDGE

3.1 Location:

The Market Street Sidehill Viaduct No. 1 is located in the Twin Peaks portion of Market Street between Short and Stanton Streets. It supports the northbound lanes of Market Street, whereas the southbound lanes are supported on grade.

3.2 Physical Description:

The viaduct is 370 ft long and includes two 15-ft end spans and seventeen 20-ft intermediate spans. It is divided into four segments by expansion joints, with each joint being located five feet from a supporting bent. The structure is 41 ft wide and supports a 31-ft wide roadway, an 8-ft wide sidewalk, and 2-ft median strip. 18-ft long approach aprons are located at each end (Figure 1).

Bents are supported on four 2-ft-diameter, cast-in-place concrete piles. The piles are spirally reinforced. Exterior piles are vertical while the interior piles are on a 1:4 batter.

The deck between bents is a 12-in. thick reinforced concrete slab. The viaduct is divided by expansion joints into one unit 110 ft long, two units 80 ft long, and one unit 100 ft long.

Most of the structure was constructed directly on grade. The ground falls away from the easterly third of the roadway, and the tops of the easterly row of piles are exposed. The maximum height of exposed pile is eight feet.

The viaduct was constructed in 1957 as part of a general widening and realignment of Market Street between Hattie Street and Corbett Avenue.

3.3 Traffic Flow:

Market Street provides the most direct route between the financial and commercial center of San Francisco and the western residential districts between Golden Gate Park and Lake Merced and therefore carries

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heavy passenger and truck traffic. A detour to the north of Twin Peaks, if required by failure of any structure on Market Street, would add about 1.6 miles to the distance across the City.

3.4 Description of Bridge Condition:

Very little of the underside of this viaduct or its supporting columns are visible for inspection. What could be seen is in good condition.

3.5 Description of Specialty Items:

The deck and bent caps are monolithic and the bent caps are cast directly on the supporting piles. The only part subject to movements are the expansion joints which are located at the quarter points in the deck slabs and at the abutments.

Expansion joints are constructed with a 12-in.-wide bearing surface across the full width of the deck. Contact surfaces are separated by two 1/16-ft thick layers of asbestos sheet packing.

Expansion joints are in fair condition, except at the south abutment where the expansion joint armor in the apron side of the joint is missing.

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4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING BRIDGE

4.1 The criteria for analysis of the existing bridge, as well as for the modified bridges, are given in Appendix A. A general summary of the appendix as it pertains to the Market Street Sidehill Viaduct No. 1 is given below:

4.1.1 The soil investigation shows that the bridge is situated on alluvium with 20-ft depth to bedrock. The Caltrans Response Spectrum II, shown in Figure 3a of Reference 3, was used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the peak ground acceleration (PGA) is 0.7g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where

U	=	Total load to be resisted
D.L.	=	Dead load
E.Q.X	=	Earthquake load in X-direction (longitudinal)
E.Q.Z	=	Earthquake load in Z-direction (transverse)
Z	=	Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Multicolumn pier (spirally reinforced)	8
Column pier connections	2

4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete f'_c = 2,500 psi

Reinforcing Steel f_y = 40,000 psi*

*The steel grade is not specified in the plans. However, based on the 1950 construction date, it is reasonable to assume that A15 Grade 40 steel was used.

5. SEISMIC ANALYSIS OF EXISTING BRIDGE

5.1 Analysis Method:

Market Street Viaduct No. 1 was analyzed for seismic forces using the computer program SEISAB, Version 1.3, written by the Engineering Computer Corporation (Reference 5). The program does not make provisions for battered columns, so the following procedures were used:

- An equivalent frame was assumed for the bents. This frame used vertical piles in lieu of battered piles, but the moment of inertia was tripled. The equivalent frame was used in the SEISAB program.
- The actual bent configuration was modeled for the FRAME2C computer program. The stiffness of this frame was computed and compared with the stiffness of the vertical frame, and member moments and shears were determined for a 100-kip lateral force.
- Total lateral force per bent was computed from the SEISAB model and then the FRAME2C results were factored up or down as required.

5.2 Modeling Assumptions:

5.2.1 The Market Street viaduct are all constructed with typical segments from 80 to 110 ft long. There is no longitudinal tension restraint between segments. There is a very small (2 in. deep) keyway providing transverse restraint. It has been neglected.

5.2.2 The computer model was based on a five span continuous concrete structure. The end spans, of 15 ft and 5 ft respectively, were assumed to be vertically supported but unrestrained in the longitudinal and transverse directions by fictitious abutments. The segment had three 20-ft long interior spans (Figure 2).

5.2.3 The ground surface under most of Viaduct No. 1 is directly under the slab, and only a few feet of the easterly columns are exposed. For the computer model, however, it was assumed that the cast-in-place concrete piles are unrestrained to a depth of 9 ft.

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5.3 Results of Analysis:

5.3.1 The predominate periods of vibration in the two directions, along with their corresponding spectral acceleration (S_a) and participation factors, are given below:

	T (sec)	S_a (g)	Participation Factor	Mode
X-Direction (Longitudinal)	0.21	1.58	4.71	1
Z-Direction	0.12	1.37	3.53	2
(Transverse)	0.10	1.29	3.17	3

5.3.2 The deflections of the structure, based on the assumptions of the computer model are:

Longitudinal (S_x)	0.62 in.
Transverse (S_z)	
Abut. #1	0.30
Bent #2	0.22
Bent #3	0.13
Bent #4	0.10
Bent #5	0.16
Abut. #6	0.18

The transverse stiffnesses of the bents as modeled from the SEISAB computer program and as actually computed by the FRAME2C program are relatively close. Therefore, the SEISAB transverse displacements are realistic.

5.3.3 The bending moments in the columns, based on the assumptions of the computer model, are:

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TRANSVERSE BENDING MOMENTS (kip-ft)

	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	287	171	127	204
Col. 1 - Top	215	129	96	153
Col. 2 - Bottom	346	207	153	246
Col. 2 - Top	341	204	151	243
Col. 3 - Bottom	330	198	147	235
Col. 3 - Top	334	200	148	238
Col. 4 - Bottom	313	187	139	223
Col. 4 - Top	249	158	117	188

LONGITUDINAL BENDING MOMENTS (kip-ft)

JOINT	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	314	526	521	314
Col. 1 - Top	0	425	415	0
Col. 2 - Bottom	314	526	521	314
Col. 2 - Top	0	425	415	0
Col. 3 - Bottom	314	526	521	314
Col. 3 - Top	0	425	415	0
Col. 4 - Bottom	314	526	521	314
Col. 4 - Top	0	425	415	0

Inspection of the SEISAB computer printout revealed that longitudinal forces created very small transverse moments and transverse forces created very small longitudinal moments. Therefore, increases due to combinations of longitudinal and transverse forces are insignificant.

5.3.4 The interaction diagram for the columns shows that the bending capacity of the columns without axial compression is 175 kip-ft. This condition occurs at the batter piles, where tension forces are offset by the weight of the structure. Moment capacities of columns supporting axial compression are higher.

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The columns are spirally reinforced and have good ductility. The force adjustment factor (Z) is 8.0.

There is no confinement reinforcement in the column to bent cap connection areas. It is assumed that there will be loss of flexural capacity in these connections due to loss of bond of the vertical column bar anchorage or shear degradation within the joint. It is further assumed that elastic moments which should be resisted at the top of the column will be redistributed to the point of fixity.

Maximum bending moments occur in the longitudinal direction at the top of bent #3 (425 kip-ft). If these moments are entirely redistributed to the point of fixity, the column base moment becomes 526 + 425/2 = 740 ft-kips. This represents a required adjustment factor (Z) of 740/175 = 4.2.

5.3.5 Displacements are acceptable. Six tenths of an inch longitudinal displacement is only 6% of the effective bearing width.

Transverse displacement is slightly restrained by a 2-in.-deep by 3-ft-wide keyway. Some damage to the keyway interface can be expected, but structural stability will not be impaired.

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6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION

No structural modification of this viaduct is required.

Related Work:

Check drainage system and perform minor repairs	$2,000
Engineering supervision and inspection	1,000
Total	$3,000

7. SUMMARY AND CONCLUSIONS

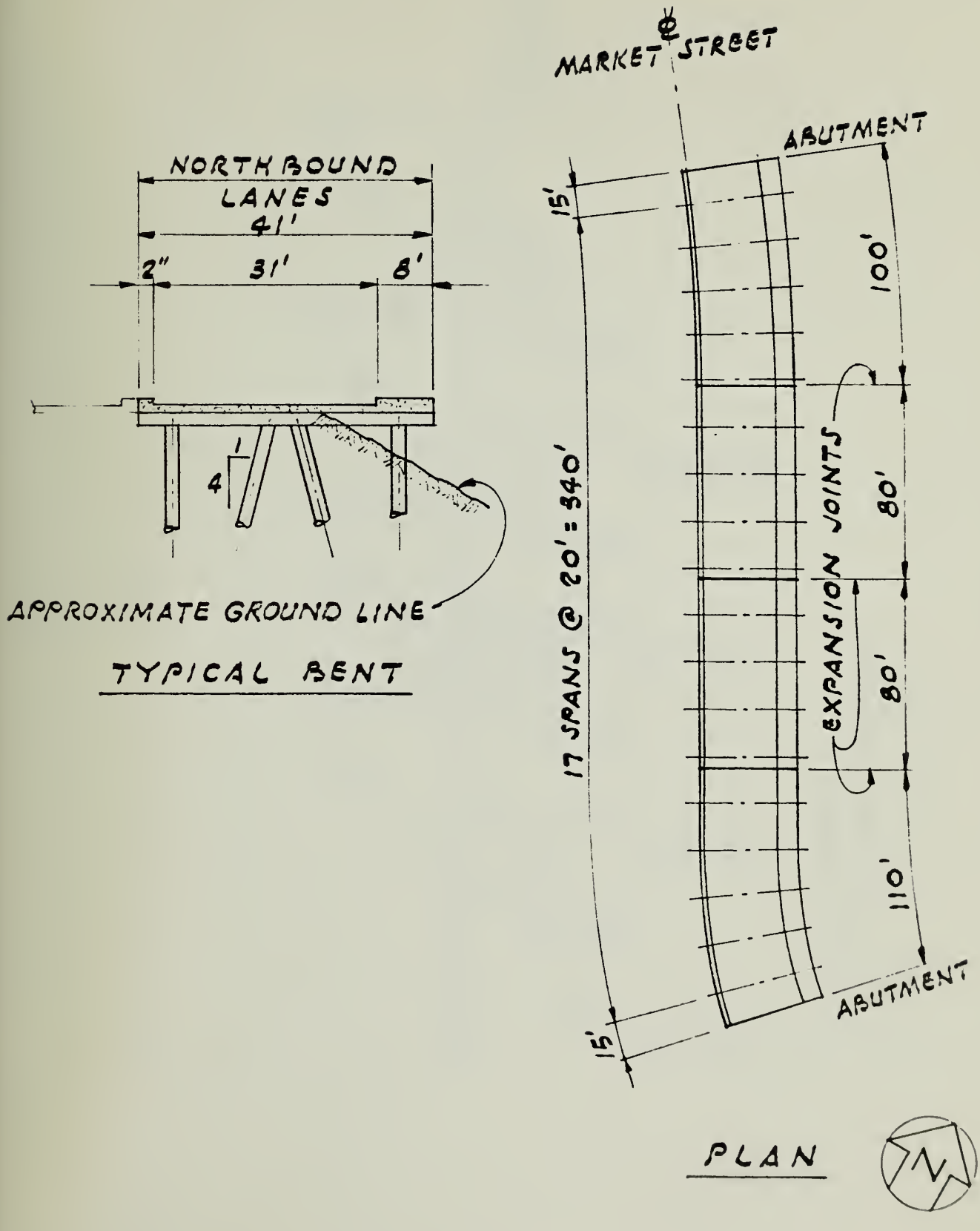
Market Street Sidehill Viaduct #1 supports the northbound lanes of Market Street between Short and Stanton Streets. It is constructed almost entirely on grade, so that the supporting cast-in-place columns are completely embedded except for the most easterly row.

The peak ground acceleration is 0.7g and the maximum spectral acceleration is 1.6g.

Displacements of the structure under the postulated earthquake forces are acceptable. Inelastic behavior is anticipated and some redistribution of moments may occur in the columns where the concrete is unconfined. Some concrete crushing may occur at the interface of the keyways in the transverse expansion joints. There should be no life safety hazard, and the structure should continue to function.

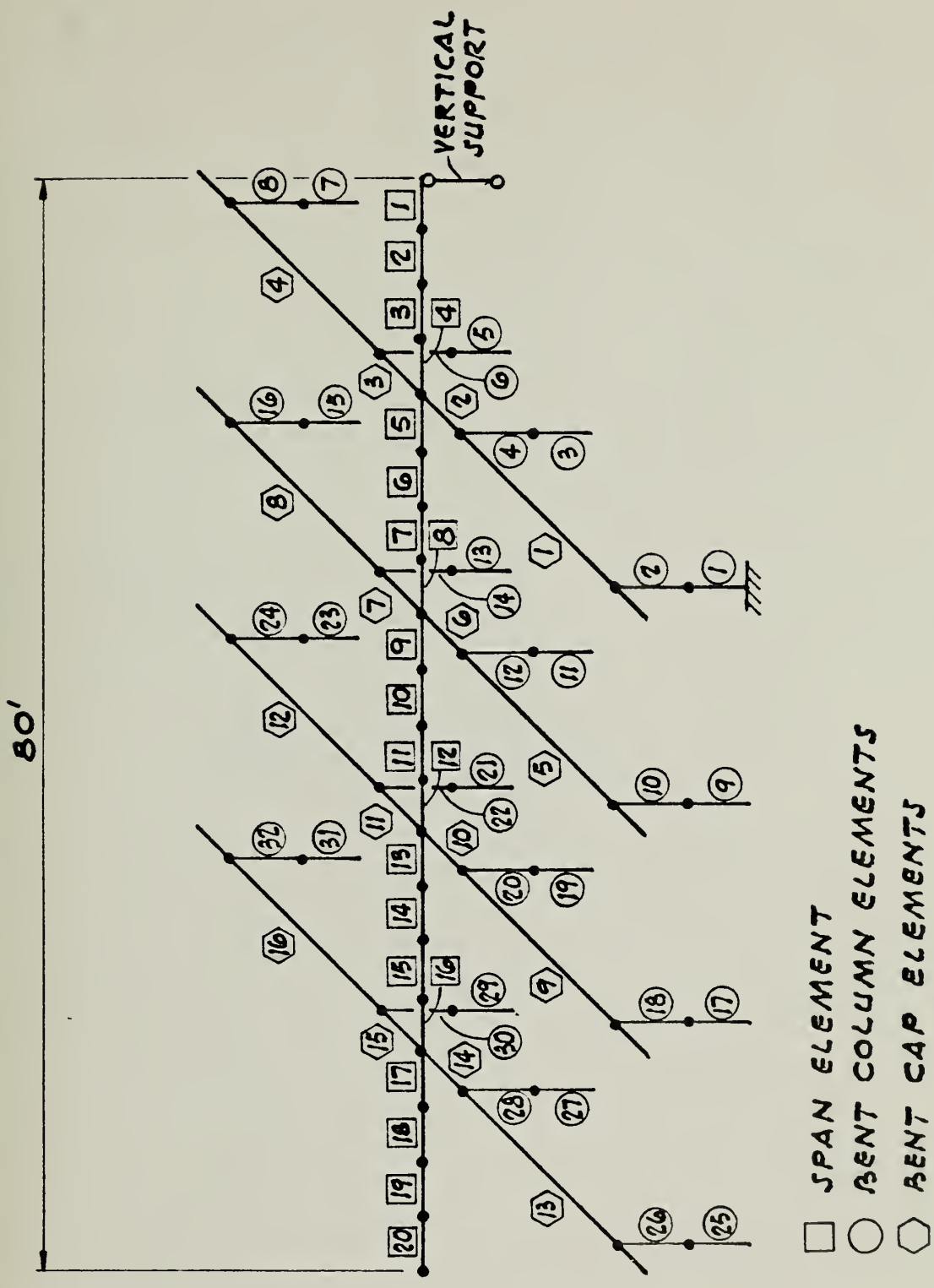
The conclusions above are dependent upon the soil under the structure remaining stable. Saturation should be avoided.

We recommend that periodic maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.



PLAN AND TYPICAL BENT
MARKET STREET VIADUCT #1
S.F. BRIDGE GROUP II A-1

FIGURE 1



COMPUTER MODEL FOR TYPICAL SEGMENT
MARKET STREET VIADUCT #1
S.F. BRIDGE GROUP II A

FIGURE 2

MARKET STREET VIADUCT #2

IIA-2

MARKET STREET SIDEHILL VIADUCT NO. 2

CONTENTS

	page
1. INTRODUCTION	IIA-2.1
2. SITE INVESTIGATION	IIA-2.2
3. BRIDGE DESCRIPTION	IIA-2.4
4. CRITERIA FOR ANALYSIS OF THE EXISTING BRIDGE	IIA-2.6
5. SEISMIC ANALYSIS OF THE EXISTING BRIDGE	IIA-2.7
6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-2.11
7. SUMMARY AND CONCLUSIONS	IIA-2.12
REFERENCES	APPENDIX C

FIGURES:

1. PLAN AND TYPICAL BENT	IIA-2.13
2. COMPUTER MODEL	IIA-2.14

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MARKET STREET SIDEHILL VIADUCT NO. 2

1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

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2. SITE INVESTIGATION

Results of the site investigation for all bridges is given in Reference 3. A general summary of the reference as it pertains to the Market Street Sidehill Viaduct No. 2 is given in this section.

This viaduct is a cast-in-place concrete vehicular bridge along Market Street between Romain Street and Morgan Alley.

Available geological information indicates that the site is underlain by undivided surficial deposits (Qu). The underlying bedrock is Franciscan Formation of the Central Highlands belt. The depth to the bedrock is estimated to be approximately 30 ft below the sloping ground. Three exploratory borings, DH-12, DH-13, and DH-14, drilled at the vicinity of the site indicate that the surficial materials consist of loose to medium dense clayey sand (SC) and sandy silt (ML), and stiff to hard silty and sandy clay (CL). Ground water level was measured at a depth ranging from 13 ft to 23 ft at the borings.

The viaduct is located at approximately 6 miles northeast of the San Andreas fault and 13 miles southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 24 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum II, shown in Appendix B, are recommended for the seismic safety evaluation.

The cohesive nature of the soil underlying the viaduct and the low ground water level suggest that liquefaction is not likely. Plate 10 of Reference 3 shows that the viaduct is located in the area of potential landslide. Our reconnaissance of the site, however, did not indicate any signs of land instability. No slides were identified at the site by available published data. We believe that the risk of landslide during earthquake is not significant. Minor downhill ground movement, however, may be induced by a major earthquake. Such movement may result in minor structural cracking. We recommend that periodical maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause

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significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.

The site is located at a great distance from the areas of potential lateral spreading and tsunamni inundation as shown on Plates 3 and 11 of Reference 3.

3. DESCRIPTION OF BRIDGE

3.1 Location:

The Market Street Sidehill Viaduct No. 2 is located in the Twin Peaks portion of Market Street between Romain Street and Morgan Alley. It supports the northbound lanes of Market Street, whereas the southbound lanes are supported on grade.

3.2 Physical Description:

The viaduct is 300 ft long and includes two 10-ft end spans and fourteen 20-ft intermediate spans. It is divided into three segments by expansion joints, with each joint being located five feet from a supporting bent. The structure is 41 ft wide and supports a 31-ft wide roadway, an 8-ft wide sidewalk, and 2-ft median strip. 18-ft long approach aprons are located at each end (Figure 1).

Bents are supported on four 2-ft-diameter, cast-in-place concrete piles. The piles are spirally reinforced. Exterior piles are vertical while the interior piles are on a 1:4 batter.

The deck between bents is a 12-in. thick reinforced concrete slab. The viaduct is divided by expansion joints into one unit 110 ft long and two units 95 ft long.

Most of the structure was constructed directly on grade. The ground falls away from the easterly third of the roadway, and the tops of the easterly row of piles are exposed. The maximum height of exposed pile is eight feet.

The viaduct was constructed in 1957 as part of a general widening and realignment of Market Street between Hattie Street and Corbett Avenue.

3.3 Traffic Flow:

Market Street provides the most direct route between the financial and commercial center of San Francisco and the western residential districts between Golden Gate Park and Lake Merced and therefore carries

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heavy passenger and truck traffic. A detour to the north of Twin Peaks, if required by failure of any structure on Market Street, would add about 1.6 miles to the distance across the City.

3.4 Description of Bridge Condition:

Very little of the underside of this viaduct or its supporting columns are visible for inspection. What could be seen is in good condition.

3.5 Description of Specialty Items:

The deck and bent caps are monolithic and the bent caps are cast directly on the supporting piles. The only part subject to movements are the expansion joints which are located at the quarter points in the deck slabs and at the abutments.

Expansion joints are constructed with a 12-in.-wide bearing surface across the full width of the deck. Contact surfaces are separated by two 1/16-ft thick layers of asbestos sheet packing.

Expansion joints are in good condition.

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4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING BRIDGE

4.1 The criteria for analysis of the existing bridge, as well as for the modified bridges, are given in Appendix A. A general summary of the appendix as it pertains to the Market Street Sidehill Viaduct No. 2 is given below:

4.1.1 The soil investigation shows that the bridge is situated on alluvium with 30-ft depth to bedrock. The Caltrans Response Spectrum II, shown in Figure 3a of Reference 3, was used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the peak ground acceleration (PGA) is 0.7g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where

U	=	Total load to be resisted
D.L.	=	Dead load
E.Q.X	=	Earthquake load in X-direction (longitudinal)
E.Q.Z	=	Earthquake load in Z-direction (transverse)
Z	=	Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Multicolumn pier (spirally reinforced)	8
Column pier connections	2

4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete	f'_c	= 2,500 psi
Reinforcing Steel	f_y	= 40,000 psi*

*The steel grade is not specified in the plans. However, based on the 1950 construction date, it is reasonable to assume that A15 Grade 40 steel was used.

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5. SEISMIC ANALYSIS OF EXISTING BRIDGE

5.1 Analysis Method:

Market Street Viaduct No. 2 was analyzed for seismic forces using the computer program SEISAB, Version 1.3, written by the Engineering Computer Corporation (Reference 5). The program does not make provisions for battered columns, so the following procedures were used:

- An equivalent frame was assumed for the bents. This frame used vertical piles in lieu of battered piles, but the moment of inertia was tripled. The equivalent frame was used in the SEISAB program.
- The actual bent configuration was modeled for the FRAME2C computer program. The stiffness of this frame was computed and compared with the stiffness of the vertical frame, and member moments and shears were determined for a 100-kip lateral force.
- Total lateral force per bent was computed from the SEISAB model and then the FRAME2C results were factored up or down as required.

5.2 Modeling Assumptions:

5.2.1 The Market Street viaduct are all constructed with typical segments from 80 to 110 ft long. There is no longitudinal tension restraint between segments. There is a very small (2 in. deep) keyway providing transverse restraint. It has been neglected.

5.2.2 The computer model was based on a five span continuous concrete structure. The end spans, of 15 ft and 5 ft respectively, were assumed to be vertically supported but unrestrained in the longitudinal and transverse directions by fictitious abutments. The segment had three 20-ft long interior spans (Figure 2).

5.2.3 The ground surface under most of Viaduct No. 2 is directly under the slab, and only a few feet of the easterly columns are exposed. For the computer model, however, it was assumed that the cast-in-place concrete piles are unrestrained to a depth of 9 ft.

5.3 Results of Analysis:

5.3.1 The predominate periods of vibration in the two directions, along with their corresponding spectral acceleration (S_a) and participation factors, are given below:

	T (sec)	S_a (g)	Participation Factor	Mode
X-Direction (Longitudinal)	0.21	1.58	4.71	1
Z-Direction	0.12	1.37	3.53	2
(Transverse)	0.10	1.29	3.17	3

5.3.2 The deflections of the structure, based on the assumptions of the computer model are:

Longitudinal (S_x) 0.62 in.

Transverse (S_z)

Abut. #1	0.30
Bent #2	0.22
Bent #3	0.13
Bent #4	0.10
Bent #5	0.16
Abut. #6	0.18

The transverse stiffnesses of the bents as modeled from the SEISAB computer program and as actually computed by the FRAME2C program are relatively close. Therefore, the SEISAB transverse displacements are realistic.

5.3.3 The bending moments in the columns, based on the assumptions of the computer model, are:

TRANSVERSE BENDING MOMENTS (kip-ft)

	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	287	171	127	204
Col. 1 - Top	215	129	96	153
Col. 2 - Bottom	346	207	153	246
Col. 2 - Top	341	204	151	243
Col. 3 - Bottom	330	198	147	235
Col. 3 - Top	334	200	148	238
Col. 4 - Bottom	313	187	139	223
Col. 4 - Top	249	158	117	188

LONGITUDINAL BENDING MOMENTS (kip-ft)

JOINT	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	314	526	521	314
Col. 1 - Top	0	425	415	0
Col. 2 - Bottom	314	526	521	314
Col. 2 - Top	0	425	415	0
Col. 3 - Bottom	314	526	521	314
Col. 3 - Top	0	425	415	0
Col. 4 - Bottom	314	526	521	314
Col. 4 - Top	0	425	415	0

Inspection of the SEISAB computer printout revealed that longitudinal forces crested very small transverse moments and transverse forces crested very small longitudinal moments. Therefore, increases due to combinations of longitudinal and transverse forces are insignificant.

5.3.4 The interaction diagram for the columns shows that the bending capacity of the columns without axial compression is 175 kip-ft. This condition occurs at the batter piles, where tension forces are offset by the weight of the structure. Moment capacities of columns supporting axial compression are higher.

The columns are spirally reinforced and have good ductility. The force adjustment factor (Z) is 8.0.

There is no confinement reinforcement in the column to bent cap connection areas. It is assumed that there will be loss of flexural capacity in these connections due to loss of bond of the vertical column bar anchorage or shear degradation within the joint. It is further assumed that elastic moments which should be resisted at the top of the column will be redistributed to the point of fixity.

Maximum bending moments occur in the longitudinal direction at the top of bent #3 (425 kip-ft). If these moments are entirely redistributed to the point of fixity, the column base moment becomes 526 + 425/2 = 740 ft-kips. This represents a required adjustment factor (Z) of 740/175 = 4.2.

5.3.5 Displacements are acceptable. Six tenths of an inch longitudinal displacement is only 6% of the effective bearing width.

Transverse displacement is slightly restrained by a 2-in.-deep by 3-ft-wide keyway. Some damage to the keyway interface can be expected, but structural stability will not be impaired.

6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION

No structural modification of this viaduct is required.

Related Work:

Check drainage system and perform minor repairs	$2,000
Engineering supervision and inspection	1,000
Total	$3,000

7. SUMMARY AND CONCLUSIONS

Market Street Sidehill Viaduct #2 supports the northbound lanes of Market Street between Romain Street and Morgan Alley. It is constructed almost entirely on grade, so that the supporting cast-in-place columns are completely embedded except for the most easterly row.

The peak ground acceleration is 0.7g and the maximum spectral acceleration is 1.6g.

Displacements of the structure under the postulated earthquake forces are acceptable. Inelastic behavior is anticipated and some redistribution of moments may occur in the columns where the concrete is unconfined. Some concrete crushing may occur at the interface of the keyways in the transverse expansion joints. There should be no life safety hazard, and the structure should continue to function.

The conclusions above are dependent upon the soil under the structure remaining stable. Saturation should be avoided.

We recommend that periodic maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.

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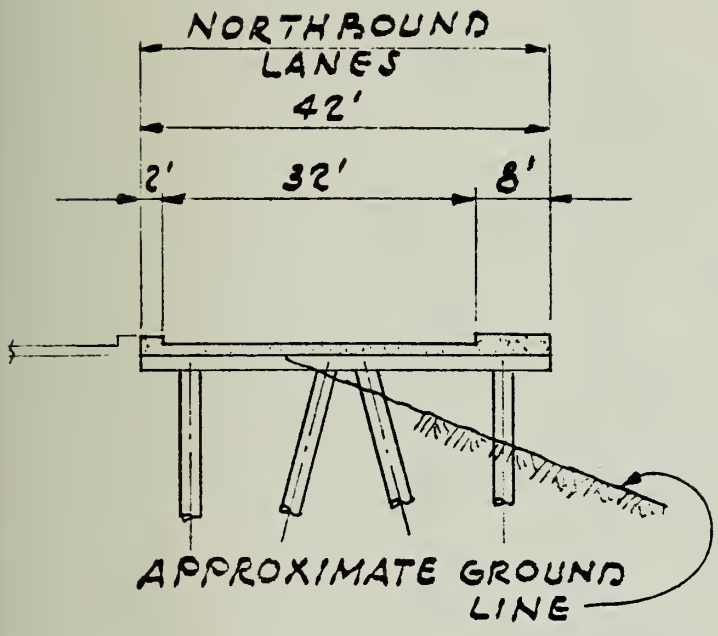
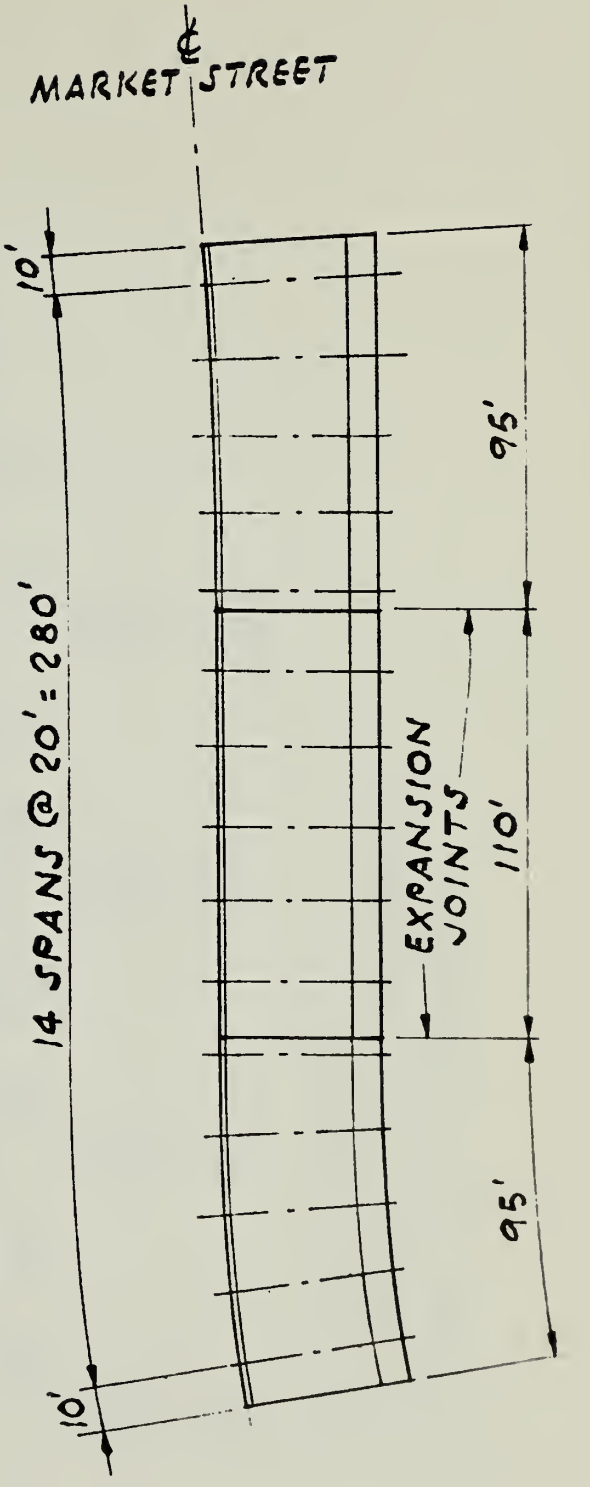


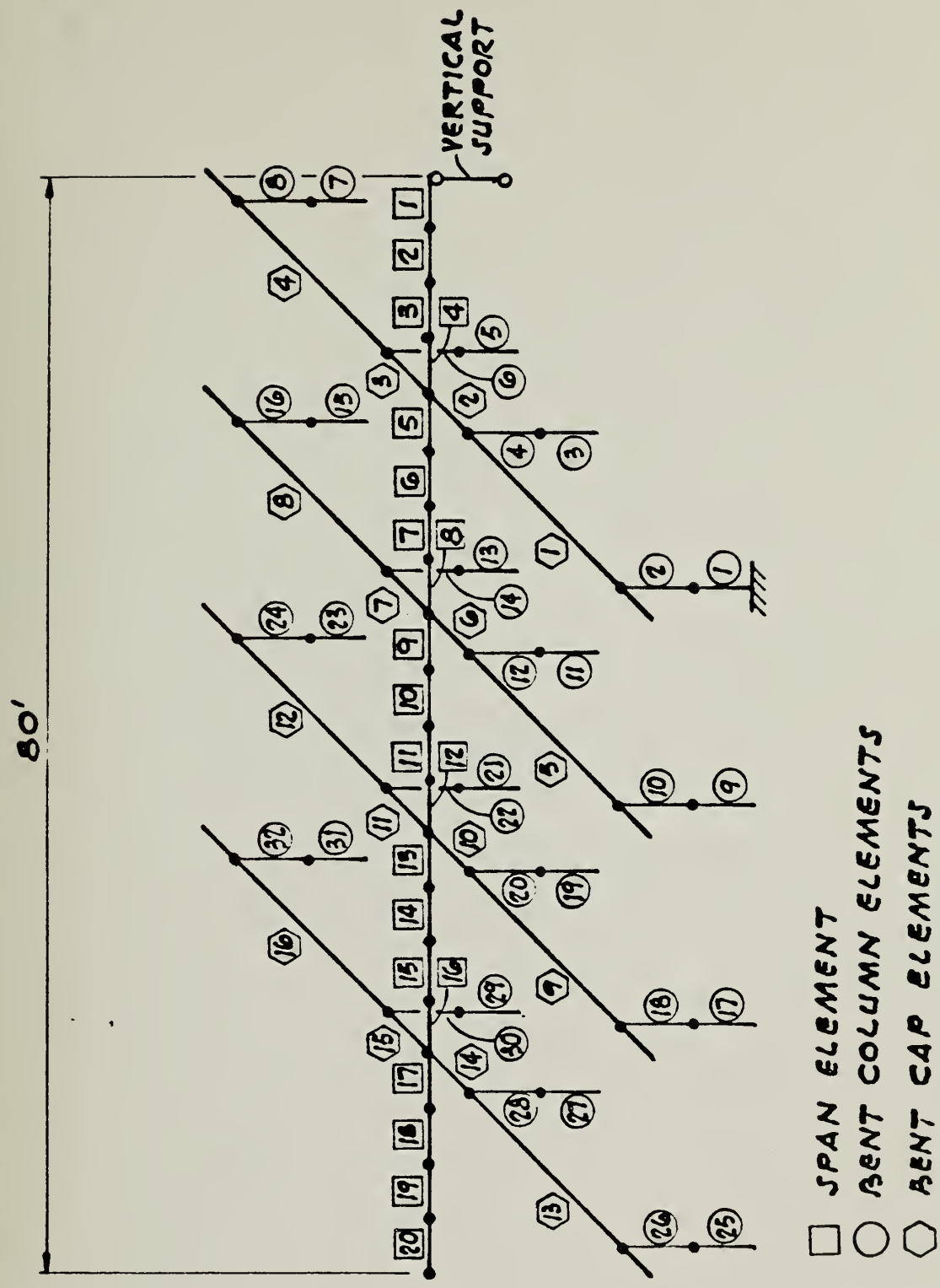
TYPICAL BENT



PLAN

PLAN AND TYPICAL BENT
MARKET STREET VIADUCT #2
S.F. BRIDGE GROUP IIA-2

FIGURE 1



COMPUTER MODEL FOR TYPICAL SEGMENT
MARKET STREET VIADUCT #2
S.F. BRIDGE GROUP II A-2

FIGURE 2

MARKET STREET VIADUCT #3

IIA-3

MARKET STREET SIDEHILL VIADUCT NO. 3

CONTENTS

		page
1.	INTRODUCTION	IIA-3.1
2.	SITE INVESTIGATION	IIA-3.2
3.	BRIDGE DESCRIPTION	IIA-3.4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE	IIA-3.8
5.	SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE	IIA-3.9
6.	RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-3.16
7.	SUMMARY AND CONCLUSIONS	IIA-3.18

REFERENCES ..... APPENDIX C

FIGURES:

1.	PLAN	IIA-3.19
2.	TYPICAL BENTS	IIA-3.20
3.	BENT CAP CONFINEMENT	IIA-3.21
4.	EXPANSION JOINT SHEAR LUGS - SH. 1	IIA-3.22
5.	EXPANSION JOINT SHEAR LUGS - SH. 2	IIA-3.23
6.	EXPANSION JOINT LONGITUDINAL RESTRAINT	IIA-3.24
7.	COMPUTER MODEL - MODIFIED STRUCTURE	IIA-3.25

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MARKET STREET SIDEHILL VIADUCT NO. 3

1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO] has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

2. SITE INVESTIGATION

Results of the site investigation for all bridges is given in Reference 3. A general summary of the reference as it pertains to the Market Street Sidehill Viaduct No. 3 is given in this section.

This viaduct is a cast-in-place concrete vehicular bridge along Market Street near 23rd Street. It is located immediately to the east of Market Street Viaduct No. 4, and the centerlines of the two viaducts are parallel.

Available geological information indicates that the site is underlain by undivided surficial deposits (Qu). The underlying bedrock is Franciscan Formation of the Central Highlands belt. The depth to the bedrock is estimated to be approximately 30 ft below the sloping ground. An exploratory boring, DH-13, was drilled at the intersection of 23rd Street and Grand View Avenue to explore the subsurface conditions. The boring was 27 ft deep, and the materials encountered consisted of approximately 4 ft of loose sandy silt (ML), underlain by approximately 23 ft of stiff to very stiff silty clay (CL). The bedrock was encountered at the depth of 27 ft. Standard blow counts of upper sandy silt (ML) were 8 per ft of penetraton, and standard blow counts of the stiff silty clay (CL) ranged from 10 to 29 per foot of penetration. The ground water was measured at the depth of 13 ft in the boring.

The viaduct is located at approximately 6 miles northeast of the San Andreas fault and 13 miles southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 24 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum II, shown in Appendix B are recommended for the seismic safety evaluation.

The cohesive nature of the soil underlying the viaduct and the low ground water level suggest that liquefaction is not likely. Plate 10 of Reference 3 shows that the viaduct is located in the area of potential landslide. Our reconnaissance of the site, however, did not indicate any signs of land instability. No slides were identified at the site by available published

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data. Geological maps at the Department of Public Works, City and County of San Francisco, indicate, however, that several small slides took place previously to the south of the site. Two of the slides were located at the vicinity of the viaduct between Elizabeth Street and 24th Street and between 24th Street and 25th Street on Grand View Avenue. These two slides occurred in 1941 and were repaired in 1941 and 1942. We believe that the risk of landslide during earthquake is not significant. Minor downhill ground movement, however, may be induced by a major earthquake. Such movement may result in minor structural cracking. We recommend that periodical maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.

The site is located at a great distance from the areas of potential lateral spreading and tsunamni inundation as shown on Plates 3 and 11 of Reference 3.

3. DESCRIPTION OF BRIDGE

3.1 Location:

The Market Street Sidehill Viaduct No. 3 is located in the Twin Peaks portion of Market Street north of 23rd Street. It supports the northbound lanes of Market Street and is adjacent to Viaduct #4 which supports the southbound lanes.

The south end of this viaduct abuts the north end of the Market Street Prestressed Viaduct.

3.2 Physical Description:

The viaduct is 560 ft long and includes two 10-ft end spans and twenty seven 20-ft intermediate spans. It is divided into six segments by expansion joints, with each joint being located five feet from a supporting bent. The structure is 42 ft wide and supports a 32-ft wide roadway, an 8-ft wide sidewalk, and 2-ft median strip. 18-ft long approach aprons are located at each end (Figure 1).

Bents are supported on 2-ft-diameter cast-in-place concrete piles. The piles are spirally reinforced. Four different types of bents are used as shown in Figure 2.

The location of the four types are given in the following schedule:

Station	Types of Bent
31 + 40 - Abutment	Type B
+ 50	Type B
+ 70	Type B
+ 90	Type B
32 + 10	Type B
+ 30	Type A
+ 35 Exp. Joint	
+ 50	Type A
+ 70	Type A
+ 90	Type A

Station	Types of Bent
33 + 10	Type A
+ 30	Type A
+ 35 Exp. Joint	
+ 50	Type A
+ 70	Type A
+ 90	Type A
34 + 10	Type C
+ 30	Type A
+ 35 Exp. Joint	
+ 50	Type C
+ 70	Type A
+ 90	Type A
35 + 10	Type D
35 + 30	Type D
+ 35 Exp. Joint	
Bents 20' o.c.	Type D
36 + 15 Exp. Joint	
Bents 20' o.c.	Type D
37 + 00 Abutment	North end - Prestressed Viaduct

The deck between bents is a 12-in. thick reinforced concrete slab. The viaduct is divided by expansion joints into one unit 85 ft long, one unit 80 ft long, three units 100 ft long, and one unit 95 feet long.

The north and south ends of this structure are constructed on grade except for the easterly third of the roadway. Between Stations 32+10 and 35+10, the viaduct crosses a shallow ravine, so that maximum exposed column heights are about twenty feet. The exposed columns are vertical. The center column of each Type A bent is supported on two batter piles.

The viaduct was constructed in 1957 as part of a general widening and realignment of Market Street between Hattie Street and Corbett Avenue.

3.3 Traffic Flow:

Market Street provides the most direct route between the financial and commercial center of San Francisco and the western residential districts between Golden Gate Park and Lake Merced and therefore carries heavy passenger and truck traffic. A detour to the north of Twin

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Peaks, if required by failure of any structure on Market Street, would add about 1.6 miles to the distance across the City.

3.4 Description of Bridge Condition:

About fifty percent of the underside of this viaduct and its supporting columns are visible for inspection. The viaduct appears to be in good condition.

3.5 Description of Specialty Items:

The deck and bent caps are monolithic and the bent caps are cast directly on the supporting piles. The only part subject to movements are the expansion joints which are located at the quarter points in the deck slabs and at the abutments.

The connections between the column top and the bent cap at the bents on either side of the expansion joints are designed to act as hinges in the direction parallel to the bridge centerline. The reinforcing provides fixity in the transverse directions. All other column to cap connections are designed to have fixity in both directions.

Expansion joints are constructed with a 12-in.-wide bearing surface across the full width of the deck. Contact surfaces are separated by two 1/16-ft thick layers of asbestos sheet packing.

Expansion joints are in good condition.

4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE

4.1 The criteria for analysis of the existing bridge, as well as for the modified bridges, are given in Appendix A. A general summary of the appendix as it pertains to the Market Street Sidehill Viaduct No. 4 is given below:

4.1.1 The soil investigation shows that the bridge is situated on alluvium with 30-ft depth to bedrock. The Caltrans Response Spectrum II, shown in Appendix B was used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the peak ground acceleration (PGA) is 0.7g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where

U	=	Total load to be resisted
D.L.	=	Dead load
E.Q.X	=	Earthquake load in X-direction (longitudinal)
E.Q.Z	=	Earthquake load in Z-direction (transverse)
Z	=	Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Multicolumn pier (spirally reinforced)	8
Column pier connections	2

4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete f'_c = 2,500 psi

Reinforcing Steel f_y = 40,000 psi*

The steel grade is not specified in the plans. However, based on the 1950 construction date, it is reasonable to assume that A15 Grade 40 steel was used.

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5. SEISMIC ANALYSIS OF EXISTING AND MODIFIED BRIDGE

5.1 Analysis Method:

5.1.1 Market Street Viaduct No. 3 was analyzed for seismic forces using the computer program SEISAB, Version 1.3 (Reference 5), written by the Engineering Computer Corporation.

5.1.2 The program does not make provisions for battered columns. On segments of the viaduct supported on battered piles (Type B bents), the following procedures were used:

- An equivalent frame was assumed for the bents. This frame used vertical piles in lieu of battered piles, but the moment of inertia was tripled. The equivalent frame was used in the SEISAB program.
- The actual bent configuration was modeled for the FRAME2C computer program. The stiffness of this frame was computed and compared with the stiffness of the vertical frame, and member moments and shears were determined for a 100-kip lateral force.
- Total lateral force per bent was computed from the SEISAB model and then the FRAME2C results were factored up or down as required.

5.2 Modeling Assumptions - Existing Structure:

5.2.1 Viaduct No. 3 is constructed with a variety of support conditions. There is no longitudinal tension restraint between segments. There is a very small (2 in. deep) shear key which provides transverse restraint. It has been neglected, so that each segment is assumed to be completely independent of the adjacent segments.

5.2.2 The computer model for the north segment is the same as was used for Viaducts 1, 2, and 4. It was based on a five span continuous concrete structure. The end spans, of 15 ft and 5 ft respectively, were assumed to be vertically supported but unrestrained in the longitudinal and transverse directions by

fictitious abutments. The segment had three 20-ft long interior spans.

The ground surface under most of the north segment is directly under the slab. For the computer model, however, it was assumed that the cast-in-place concrete piles are unrestrained to a depth of 9 ft.

5.2.3 The two segments between Stations 32 + 35 and 34 + 35 span a shallow ravine. The bents are supported on three vertical columns which are exposed for a maximum height of 20 feet. For the computer model one segment was used. Column heights varied from 10 feet at one end of the segment to 25 feet at the other. The segment included five 20-foot intermediate spans, one 5-foot end span, and one 15-foot end span. Fictitious abutments provided vertical supports but no longitudinal restraint. One abutment was assumed to provide no transverse restraint while the other abutment was assumed to provide full transverse restraint.

5.2.4 The model used on the north segment was considered to be appropriate for the southerly three segments.

5.3 Results of Analysis:

5.3.1 The predominate periods of vibration of the north segment, along with their corresponding spectral acceleration (S_a) and participation factors, are given below:

	T (sec)	S_a (g)	Participation Factor	Mode
X-Direction (Longitudinal)	0.21	1.58	4.71	1
Z-Direction	0.12	1.37	3.53	2
(Transverse)	0.10	1.29	3.17	3

5.3.2 The deflections of the north segment, based on the assumptions of the computer model are:

Longitudinal (S_x)	0.62 in.
Transverse (S_z)	
Abut. #1	0.30
Bent #2	0.22
Bent #3	0.13
Bent #4	0.10
Bent #5	0.16
Abut. #6	0.18

The transverse stiffnesses of the bents as modeled from the SEISAB computer program and as actually computed by the FRAME2C program are relatively close. Therefore, the SEISAB transverse displacements are realistic.

5.3.3 The bending moments in the columns of the north segment, based on the assumptions of the computer model, are:

TRANSVERSE BENDING MOMENTS (kip-ft)

	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	287	171	127	204
Col. 1 - Top	215	129	96	153
Col. 2 - Bottom	346	207	153	246
Col. 2 - Top	341	204	151	243
Col. 3 - Bottom	330	198	147	235
Col. 3 - Top	334	200	148	238
Col. 4 - Bottom	313	187	139	223
Col. 4 - Top	249	158	117	188

LONGITUDINAL BENDING MOMENTS (kip-ft)

JOINT	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	314	526	521	314
Col. 1 - Top	0	425	415	0
Col. 2 - Bottom	314	526	521	314
Col. 2 - Top	0	425	415	0
Col. 3 - Bottom	314	526	521	314
Col. 3 - Top	0	425	415	0
Col. 4 - Bottom	314	526	521	314
Col. 4 - Top	0	425	415	0

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Inspection of the SEISAB computer printout revealed that longitudinal forces created very small transverse moments and transverse forces created very small longitudinal moments. Therefore, increases due to combinations of longitudinal and transverse forces are insignificant.

5.3.4 The interaction diagram for the columns shows that the bending capacity of the columns without axial compression is 175 kip-ft. This condition occurs at the batter piles, where tension forces are offset by the weight of the structure. Moment capacities of columns supporting axial compression are higher.

The columns are spirally reinforced and have good ductility. The force adjustment factor (Z) is 8.0.

There is no confinement reinforcement in the column to bent cap connection areas. It is assumed that there will be loss of flexural capacity in these connections due to loss of bond of the vertical column bar anchorage or shear degradation within the joint. It is further assumed that elastic moments which should be resisted at the top of the column will be redistributed to the point of fixity.

Maximum bending moments in the computer model occur in the longitudinal direction at the top of bent #3 (425 kip-ft). If these moments are entirely redistributed to the point of fixity, the column base moment becomes 526 + 425/2 = 740 ft-kips. This represents a required adjustment factor (Z) of 740/175 = 4.2.

5.3.5 Displacements of the north segment are acceptable. Six-tenths of an inch longitudinal displacement is only 6% of the effective bearing width.

Transverse displacement is slightly restrained by a 2-in.-deep by 3-ft-wide keyway. Some damage to the keyway interface can be expected, but structural stability will not be impaired.

5.3.6 The predominate period of vibration of a segment between Stations 32 + 35 and 34 + 35, along with its corresponding spectral accelerations (S_a) and participation factors, are given below:

	T (sec)	S_a (g)	Participation Factor	Mode
X-Direction (Longitudinal)	0.38	1.56	5.38	2
Z-Direction (Transverse)	0.56	1.36	4.29	1
	0.21	1.59	3.28	3

5.3.7 The deflections of the segment, based on the assumptions of the computer model, are:

Longitudinal (S_x)	2.29 in.
Transverse (S_z)	
Abut. #1	6.13
Bent #2	5.06
Bent #3	3.61
Bent #4	2.26
Bent #5	1.00
Abut. #6	0.90
Abut. #7	0.00

5.3.8 Displacements are unacceptable and remedial measures will be required.

5.4 Modification to Computer Model:

5.4.1 The modified viaduct will have shear keys to prevent transverse differential displacements at the expansion joints. Longitudinal differential displacement will also be restrained. To reflect these modifications, the computer model was revised as follows:

- The revised model includes two segments with transverse and longitudinal couplings at the expansion joints.

- The abutments were assigned transverse spring stiffnesses to allow for flexibility of adjacent segments.

5.5 Results of Analysis of Modified Model:

5.5.1 The predominant periods of vibration of the two segments between stations 32 + 35 and 34 + 35, acting together, along with their corresponding spectral accelerations (S_a) and participation factors, are given below:

	T (sec)	S_a (g)	Participation Factor	Mode
X-Direction (Longitudinal)	0.48	1.57	7.44	2
Z-Direction (Transverse)	0.43	1.54	6.06	1
	0.24	1.61	2.77	3
	0.19	1.56	3.44	4

5.5.2 The deflections of the two segments, based on the assumptions of the modified computer model, are:

Longitudinal (S_x)	2.44 in.
Transverse (S_z)	
Abut. #1	1.02
Bent #2	0.72
Bent #3	1.14
Bent #4	1.97
Bent #5	2.89
Bent #6	3.86
Hinge	4.10
Bent #7	3.38
Bent #8	2.44
Bent #9	1.52
Bent #10	0.76
Abut. #11	0.71

5.5.3 The bending moments in the columns of the two segment groups, based on the assumptions of the computer model, are:

TRANSVERSE BENDING MOMENTS (kip-ft)

	Col. 1 Bottom	Col. 1 Top	Col. 2 Bottom	Col. 2 Top	Col. 3 Bottom	Col. 3 Top
Bent 2	362	283	441	411	362	283
Bent 3	497	400	588	551	497	400
Bent 4	747	609	880	833	747	609
Bent 5	856	715	992	947	856	715
Bent 6	752	639	849	808	752	639
Bent 7	471	398	521	484	471	398
Bent 8	522	443	596	570	522	443
Bent 9	662	529	807	769	662	528
Bent 10	524	386	680	635	524	386

LONGITUDINAL BENDING MOMENTS (kip-ft)

	Col. 1 Bottom	Col. 1 Top	Col. 2 Bottom	Col. 2 Top	Col. 3 Bottom	Col. 3 Top
Bent 2	895	0	821	0	895	0
Bent 3	1107	845	982	720	1107	845
Bent 4	1029	854	920	746	1029	854
Bent 5	764	613	680	530	764	613
Bent 6	315	0	288	0	315	0
Bent 7	218	0	199	0	218	0
Bent 8	574	489	516	432	574	489
Bent 9	1183	974	1055	847	1183	974
Bent 10	1678	1105	1458	886	1678	1105

5.5.4 Abutment and Hinge Forces:

Forces in the abutments and hinges of the computer model are:

Abut #1	Transverse Shear	192 kips
	Longitudinal Tension	0
Hinge	Transverse Shear	175
	Longitudinal Tension	102
Abut. #11	Transverse Shear	266
	Longitudinal Tension	0

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5.5.5 Interpretation of Computer Results:

- The plastic moment capacity of the 24" round spirally reinforced columns is 175-kip-ft. The columns are spirally reinforced, so that a force adjustments factor (Z) of 8.0 is justified in the columns. An elastic moment of 8.0 x 175 = 1400-kip-ft is acceptable.

  In order to justify a "Z" factor greater than 3.0 at the column-bent cap connection, it is necessary to confine the reinforcement within the connection. This confinement is secured by clamping structural channels against the faces of the bent cap.

- Out of phase transverse displacement of the hinges (expansion joints) must be prevented. An average of the transverse forces at the three joints (210 kips) is considered as a safe level of design for the shear lugs.

- Longitudinal displacements are restrainted by the compressive limitations of the expansion joints. However, it is desirable to limit differential longitudinal displacement as much as possible. The installation of restrainer cables across the joints is recommended.

6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION

6.1 The following items of work are recommended to insure stability in the event of a major earthquake.

6.1.1 Provide confinement for the column to bent cap connections at the following locations:

Station: 32 + 70
32 + 90
33 + 10
33 + 70
33 + 90
34 + 10
34 + 70
34 + 90
35 + 10

See Figure 3 for confinement details.

6.1.2 Provide transverse shear lugs at expansion joints and Stations 32+35, 33+35, and 34+35. These shear lugs are shown in Figures 4 and 5.

6.1.3 Install tension restraining cables across the expansion joints at Stations 32+35, 33+35, and 34+35. The restraining cables are shown in Figure 6.

6.2 Cost Estimate:

The preliminary cost estimate for the proposed strengthening is given below:

Description	Quantity	Unit	Unit Price	Total
Mobilization		LS		$5,000
Remove & Replace Chain Link Fence	320	l.f.	3.00	960
Confine Columnar Tops:				
Drill 1'4" hole	108	ea.	50.00	5,400
H.S. Bolts 1"x44"	108	ea.	20.00	2,160
Misc. Metal	12000	lb.	1.50	18,000
Conc. Encasement	270	c.f.	70.00	18,900
Scaffold & forms		LS		15,000
Shear Lugs:				
Drill & Install Bolts	72	ea.	35.00	2,520
Misc. Metal	4040	lb.	5.00	20,200
Longitudinal Restraints Drill 1" hole	36	ea.	50.00	1,800
Strand Units	18	ea.	100.00	1,800
Grout Strands	36	ea.	30.00	1,080
Subtotal				$92,820
Contractors OH&P (20%)				18,560
				$111,380
Contingency (20%)				22,280
Total				$133,660
Engineering				27,000
Escalation (8%)				10,700
		TOTAL		$171,360

Related Work:

Check drainage system and perform minor repairs		$2,000
Engineering supervision and inspection		1,000
	TOTAL	$3,000

7. SUMMARY AND CONCLUSIONS

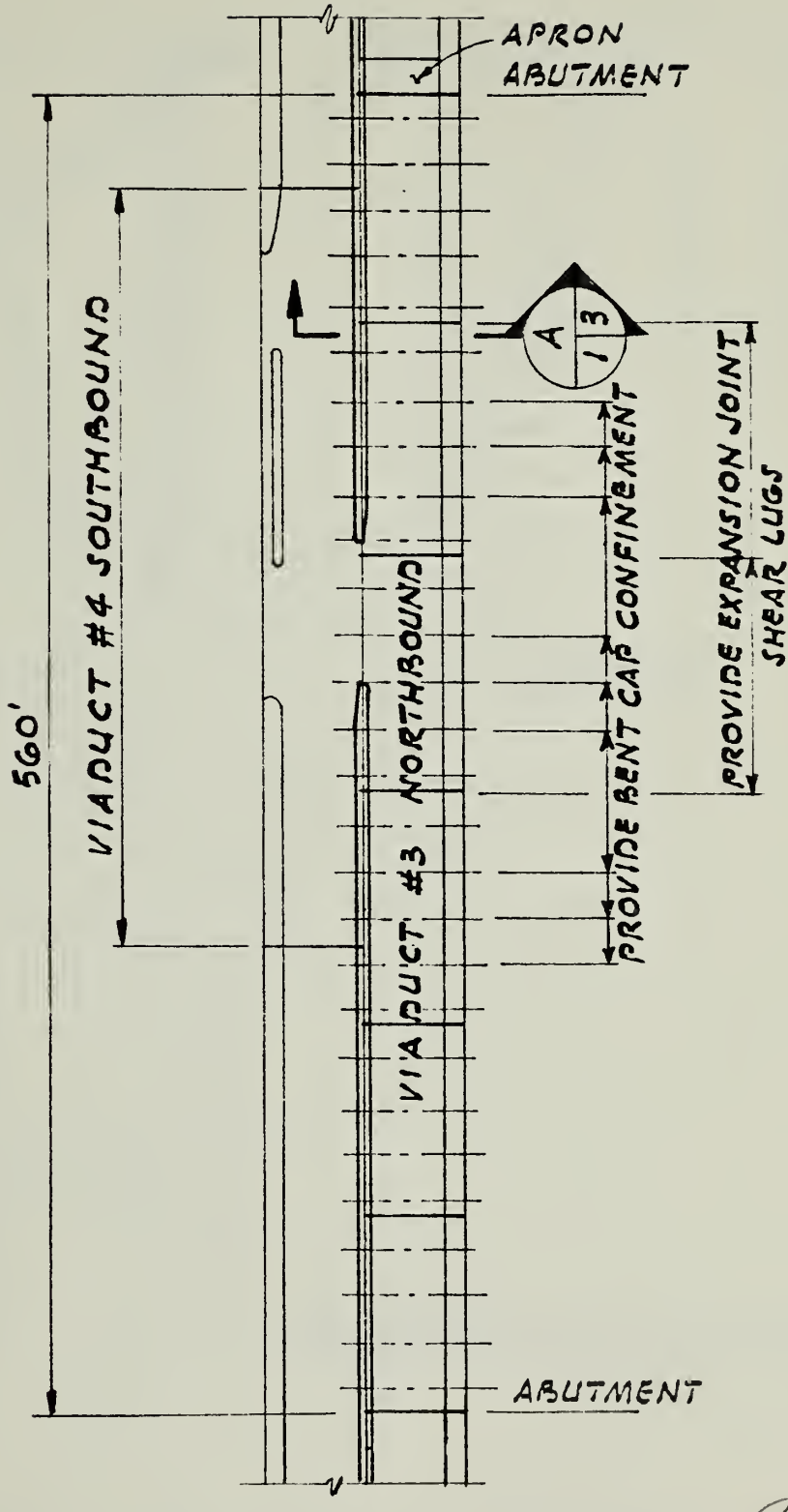
Market Street Viaduct #3 supports the northbound lanes of Market Street just north of 23rd Street. The north 80 feet and the south 160 feet of the structure are supported almost entirely on grade. The center portion crosses a shallow ravine on columns up to 20 feet in height.

The peak ground acceleration is 0.7g and the maximum spectral acceleration is 1.6g.

Modifications, as described in Section 6, are necessary to insure ductile behavior of column-bent cap connections and to reduce differential displacements at expansion joints to acceptable levels. Displacements of the modified structure under the postulated earthquake forces are acceptable. Inelastic behavior is anticipated and some redistribution of moments may occur in the columns where the concrete is unconfined. Some concrete crushing may occur at the interface of the keyways in the transverse expansion joints. There should be no life safety hazard, and the structure should continue to function.

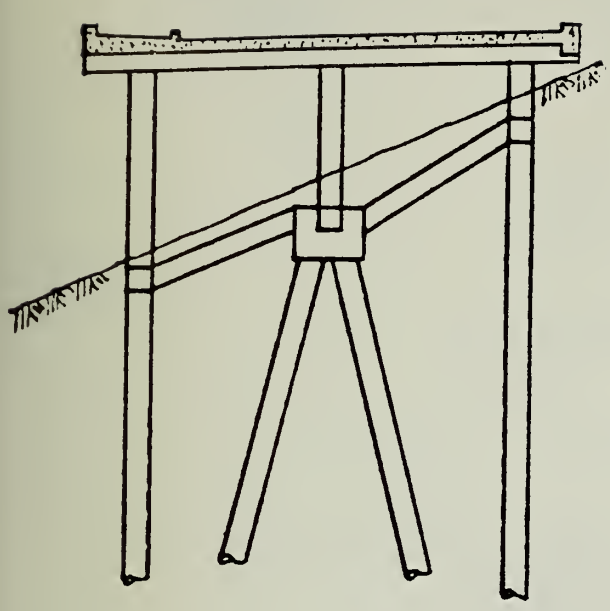
The conclusions above are dependent upon the soil under the structure remaining stable. Saturation should be avoided.

We recommend that periodic maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.



PLAN
MARKET STREET VIADUCT #3
S.F. BRIDGE GROUP II A-3

FIGURE 1



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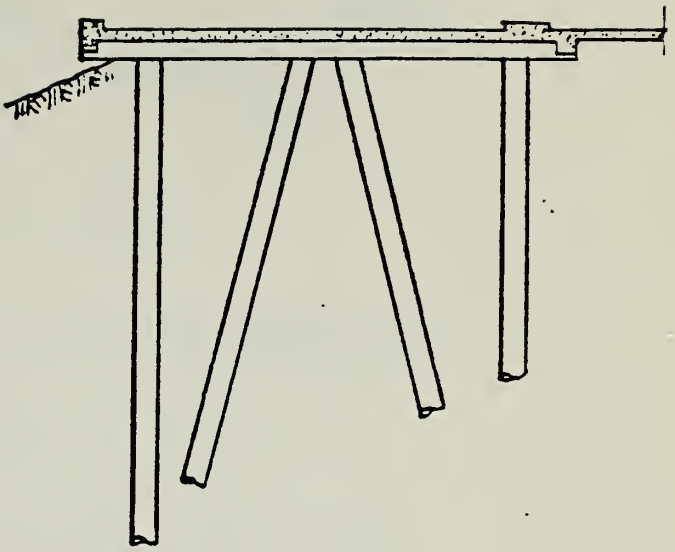
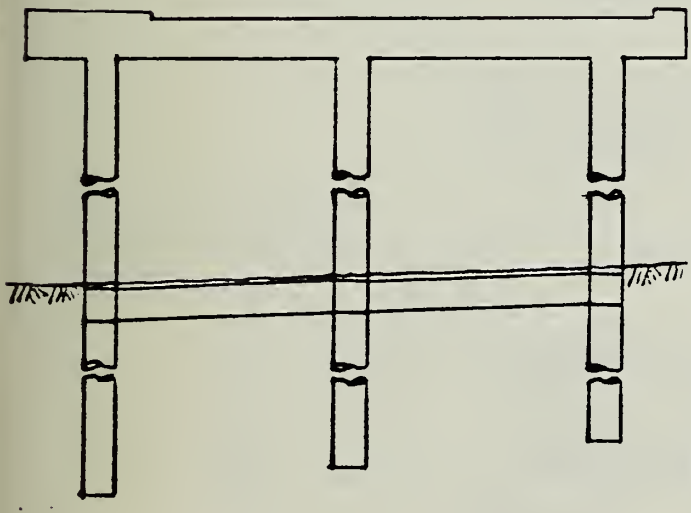


TYPE B



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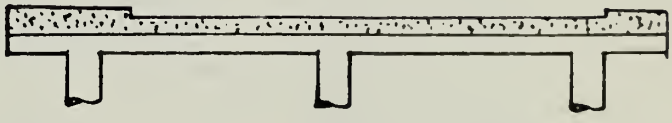
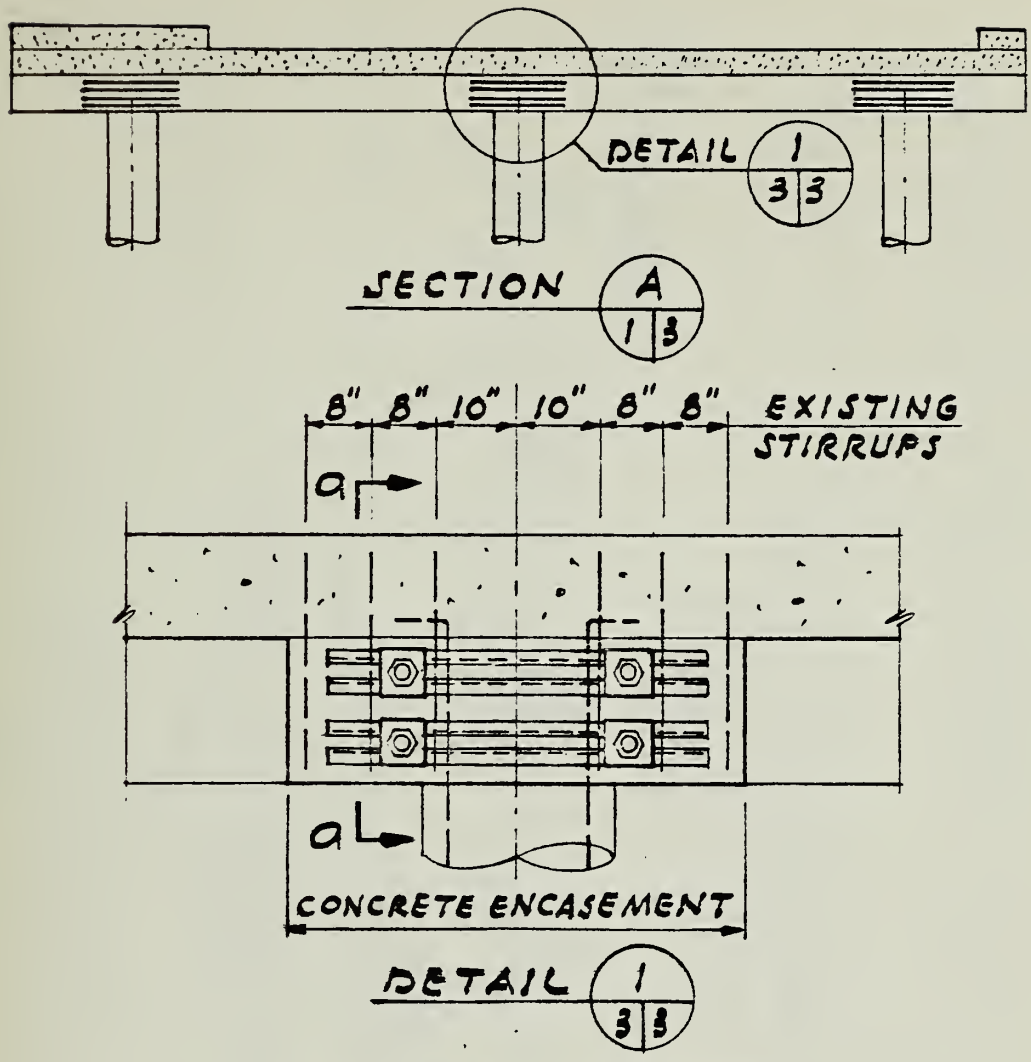


TYPE D

TYPICAL BENTS
MARKET STREET VIADUCT NO. 3
S.F. BRIDGE GROUP IIA-3

FIGURE 2



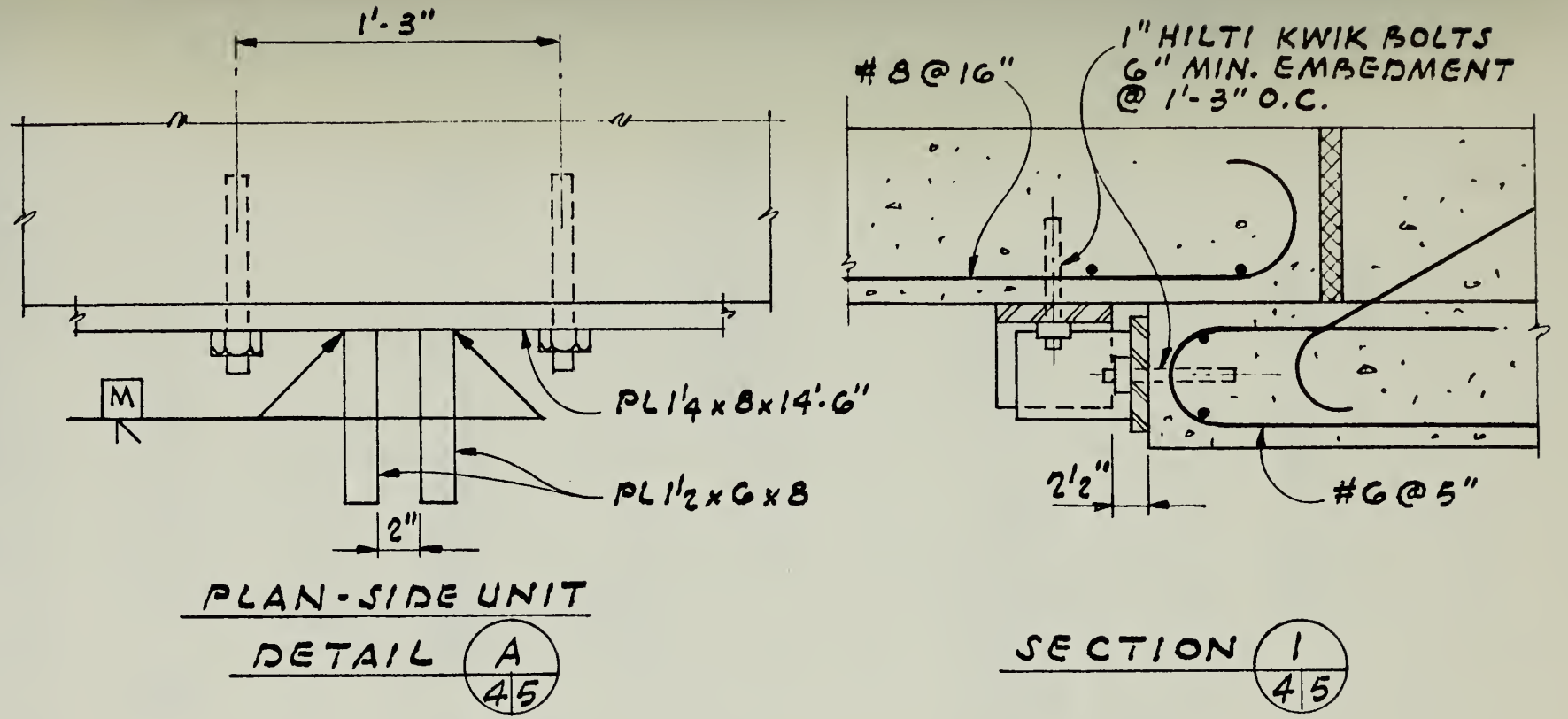
Section a-a

BENT CAP CONFINEMENT
MARKET STREET VIADUCT #3
S.F. BRIDGE GROUP IIA-3

FIGURE 3

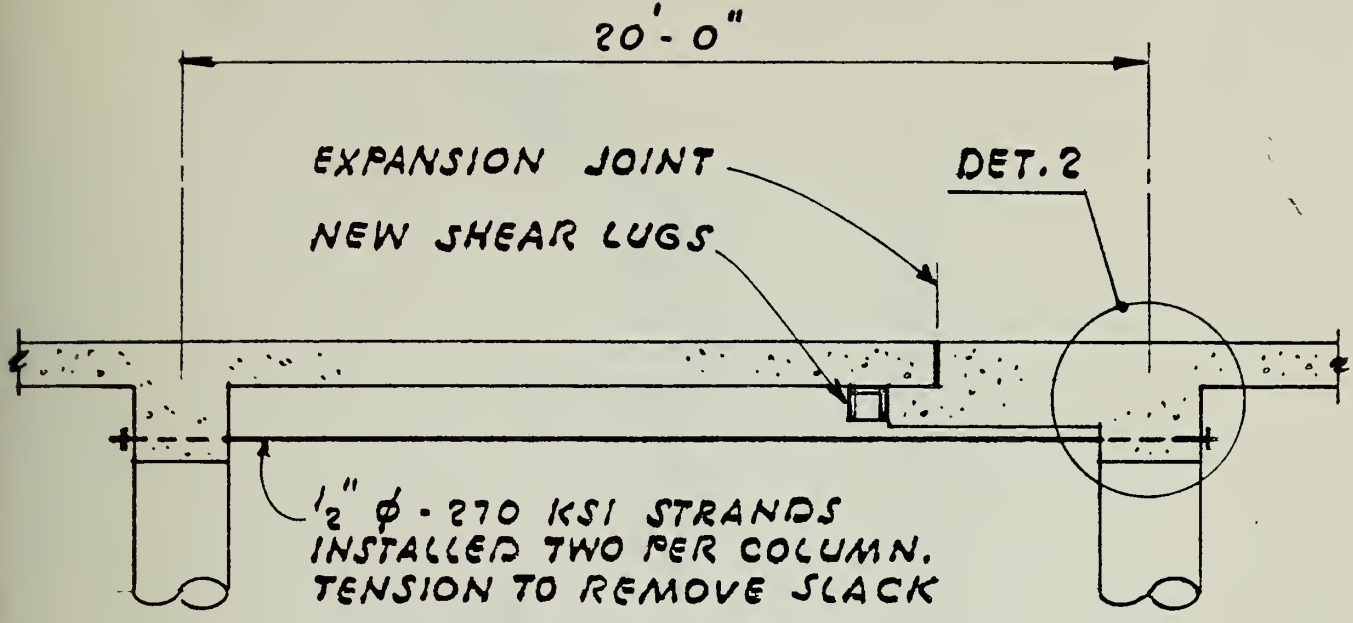
EXPANSION JOINT SHEAR LUGS
MARKET STREET VIADUCT #3
S.F. BRIDGE GROUP II A-3

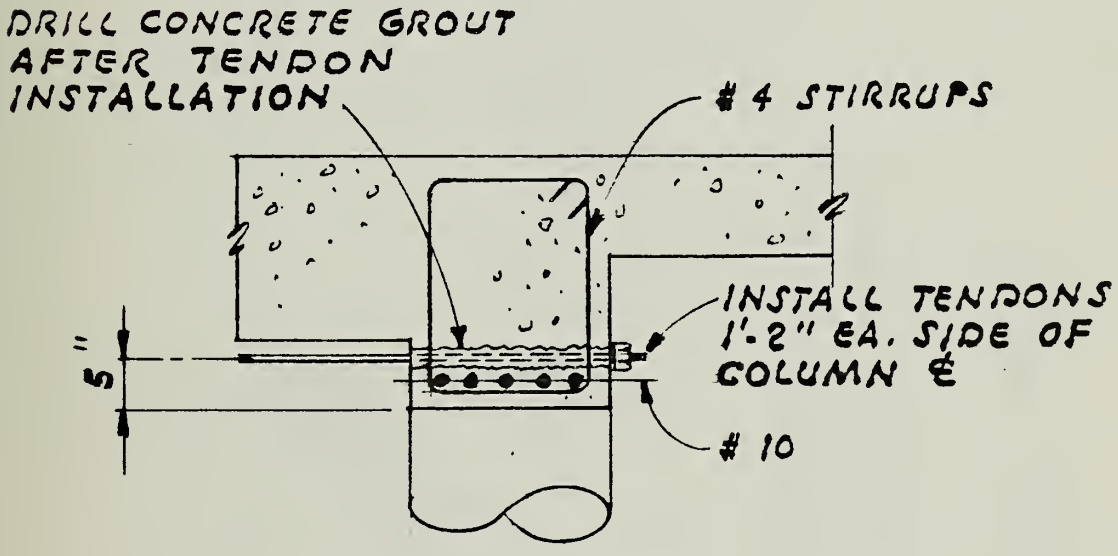
FIGURE 4



EXPANSION JOINT SHEAR LUGS
MARKET STREET VIADUCT #3
S.F. BRIDGE GROUP II A-3

FIGURE 5





DETAIL 2

EXPANSION JOINT LONGITUDINAL RESTRAINT
MARKET STREET VIADUCT #3
S.F. BRIDGE GROUP IIA-3

FIGURE 6

COMMUTER MODEL - MODIFIED STRUCTURE
MARKET STREET VIADUCT # 3
S.F. BRIDGE GROUP II A-3

FIGURE 7

MARKET STREET VIADUCT #4

IIA-4

MARKET STREET SIDEHILL VIADUCT NO. 4

CONTENTS

		page
1.	INTRODUCTION	IIA-4.1
2.	SITE INVESTIGATION	IIA-4.2
3.	BRIDGE DESCRIPTION	IIA-4.4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING BRIDGE	IIA-4.6
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE	IIA-4.7
6.	RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-4.12
7.	SUMMARY AND CONCLUSIONS	IIA-4.13

REFERENCES ..... APPENDIX C

FIGURES:

1.	PLAN	IIA-4.14
2.	COMPUTER MODEL	IIA-4.15

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1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

2. SITE INVESTIGATION

Results of the site investigation for all bridges is given in Reference 3. A general summary of the reference as it pertains to the Market Street Sidehill Viaduct No. 4 is given in this section.

This viaduct is a cast-in-place concrete vehicular bridge along Market Street near 23rd Street. It is located immediately to the west of Market Street Viaduct No. 3, and the centerlines of the two viaducts are parallel.

Available geological information indicates that the site is underlain by undivided surficial deposits (Qu). The underlying bedrock is Franciscan Formation of the Central Highlands belt. The depth to the bedrock is estimated to be approximately 30 ft below the sloping ground. An exploratory boring, DH-13, was drilled at the intersection of 23rd Street and Grand View Avenue to explore the subsurface conditions. The boring was 27 ft deep, and the materials encountered consisted of approximately 4 ft of loose sandy silt (ML), underlain by approximately 23 ft of stiff to very stiff silty clay (CL). The bedrock was encountered at the depth of 27 ft. Standard blow counts of upper sandy silt (ML) were 8 per ft of penetraton, and standard blow counts of the stiff silty clay (CL) ranged from 10 to 29 per foot of penetration. The ground water was measured at the depth of 13 ft in the boring.

The viaduct is located at approximately 6 miles northeast of the San Andreas fault and 13 miles southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 24 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum II, shown in Appendix B, are recommended for the seismic safety evaluation.

The cohesive nature of the soil underlying the viaduct and the low ground water level suggest that liquefaction is not likely. Plate 10 of Reference 3 shows that the viaduct is located in the area of potential landslide. Our reconnaissance of the site, however, did not indicate any signs of land instability. No slides were identified at the site by available published

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data. Geological maps at the Department of Public Works, City and County of San Francisco, indicate, however, that several small slides took place previously to the south of the site. Two of the slides were located at the vicinity of the viaduct between Elizabeth Street and 24th Street and between 24th Street and 25th Street on Grand View Avenue. These two slides occurred in 1941 and were repaired in 1941 and 1942. We believe that the risk of landslide during earthquake is not significant. Minor downhill ground movement, however, may be induced by a major earthquake. Such movement may result in minor structural cracking. We recommend that periodical maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.

The site is located at a great distance from the areas of potential lateral spreading and tsunamni inundation as shown on Plates 3 and 11 of Reference 3.

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3. DESCRIPTION OF BRIDGE

3.1 Location:

The Market Street Sidehill Viaduct No. 4 is located in the Twin Peaks portion of Market Street north of 23rd Street. It supports the southbound lanes of Market Street and is adjacent to Viaduct #3 which supports the northbound lanes.

3.2 Physical Description:

The viaduct is 300 ft long and includes two 10-ft end spans and fourteen 20-ft intermediate spans. It is divided into three segments by expansion joints, with each joint being located five feet from a supporting bent. The structure is 42 ft wide and supports a 32-ft wide roadway, an 8-ft wide sidewalk, and 2-ft median strip. 18-ft long approach aprons are located at each end.

Bents are supported on 2-ft-diameter, cast-in-place concrete piles. The piles are spirally reinforced. In the following schedule, three pile bents have three vertical piles. At the four pile bents, the two exterior piles are vertical while the two interior piles are on a 1:4 batter.

Station	No. of Piles
32 + 00 - Abutment	4
+ 10	3
+ 30	4
+ 50	3
+ 70	4
+ 90	4
+ 95 - Exp. Joint	
33 + 10	4
+ 30	4
+ 50	3
+ 70	4
+ 90	3
34 + 05 - Exp. Joint	
+ 10	4
+ 30	3
+ 50	3
+ 70	3
+ 90	3
35 + 00 - Abutment	3

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The deck between bents is a 12-in. thick reinforced concrete slab. The viaduct is divided by expansion joints into two units 95 ft long and one unit 110 ft long.

Much of the structure was constructed directly on grade. The ground falls away from the easterly third of the roadway, and the tops of the easterly row of piles are exposed. The maximum height of exposed pile is eight feet.

The viaduct was constructed in 1957 as part of a general widening and realignment of Market Street between Hattie Street and Corbett Avenue.

3.3 Traffic Flow:

Market Street provides the most direct route between the financial and commercial center of San Francisco and the western residential districts between Golden Gate Park and Lake Merced and therefore carries heavy passenger and truck traffic. A detour to the north of Twin Peaks, if required by failure of any structure on Market Street, would add about 1.6 miles to the distance across the City.

3.4 Description of Bridge Condition:

Very little of the underside of this viaduct or its supporting columns are visible for inspection. What could be seen is in good condition.

3.5 Description of Specialty Items:

The deck and bent caps are monolithic and the bent caps are cast directly on the supporting piles. The only part subject to movements are the expansion joints which are located at the quarter points in the deck slabs and at the abutments.

Expansion joints are constructed with a 12-in.-wide bearing surface across the full width of the deck. Contact surfaces are separated by two 1/16-ft thick layers of asbestos sheet packing.

Expansion joints are in good condition.

4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING BRIDGE

4.1 The criteria for analysis of the existing bridge, as well as for the modified bridges, are given in Appendix A. A general summary of the appendix as it pertains to the Market Street Sidehill Viaduct No. 4 is given below:

4.1.1 The soil investigation shows that the bridge is situated on alluvium with 30-ft depth to bedrock. The Caltrans Response Spectrum II, shown in Appendix B, was used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the peak ground acceleration (PGA) is 0.7g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where
- U = Total load to be resisted
- $D.L.$ = Dead load
- $E.Q.X$ = Earthquake load in X-direction (longitudinal)
- $E.Q.Z$ = Earthquake load in Z-direction (transverse)
- Z = Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Multicolumn pier (spirally reinforced)	8
Column pier connections	2

4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete f'_c = 2,500 psi

Reinforcing Steel f_y = 40,000 psi*

*The steel grade is not specified in the plans. However, based on the 1950 construction date, it is reasonable to assume that A15 Grade 40 steel was used.

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5. SEISMIC ANALYSIS OF EXISTING BRIDGE

5.1 Analysis Method:

Market Street Viaduct No. 4 was analyzed for seismic forces using the computer program SEISAB, Version 1.3, written by the Engineering Computer Corporation (Reference 5). The program does not make provisions for battered columns, so the following procedures were used for the bents with battered piles:

- An equivalent frame was assumed for the bents. This frame used vertical piles in lieu of battered piles, but the moment of inertia was tripled. The equivalent frame was used in the SEISAB program.
- The actual bent configuration was modeled for the FRAME2C computer program. The stiffness of this frame was computed and compared with the stiffness of the vertical frame, and member moments and shears were determined for a 100-kip lateral force.
- Total lateral force per bent was computed from the SEISAB model and then the FRAME2C results were factored up or down as required.

5.2 Modeling Assumptions:

5.2.1 The Market Street viaducts are all constructed with typical segments from 80 to 110 ft long. There is no longitudinal tension restraint between segments. There is a very small (2 in. deep) keyway providing transverse restraint. It has been neglected.

5.2.2 The computer model was based on a five span continuous concrete structure. The end spans, of 15 ft and 5 ft respectively, were assumed to be vertically supported but unrestrained in the longitudinal and transverse directions by fictitious abutments. The segment had three 20-ft long interior spans. The center of mass is approximately 5 ft from the center of rigidity.

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5.2.3 The ground surface under most of Viaduct No. 4 is directly under the slab, and only a few feet of the easterly columns are exposed. For the computer model, however, it was assumed that the cast-in-place concrete piles are unrestrained to a depth of 9 ft.

5.2.4 The center segment of Viaduct #4 differs from the computer model as follows:

- It has five bents intead of four
- It is symmetrical with respect to mass
- It is asymmetrical with respect to rigidity because the batter pile bents are not symmetrically placed

5.3 Results of Analysis:

5.3.1 The predominate periods of vibration in the three direction, along with their corresponding spectral acceleration (S_a) and participation factors, are given below:

	T (sec)	S_a (g)	Participation Factor	Mode
X-Direction (Longitudinal)	0.21	1.58	4.71	1
Z-Direction	0.12	1.37	3.53	2
(Transverse)	0.10	1.29	3.17	3

5.3.2 The deflections of the structure, based on the assumptions of the computer model are:

Longitudinal (S_x)	0.62 in.
Transverse (S_z)	
Abut. #1	0.30
Bent #2	0.22
Bent #3	0.13
Bent #4	0.10
Bent #5	0.16
Abut. #6	0.18

The transverse stiffnesses of the bents as modeled from the SEISAB computer program and as actually computed by the FRAME2C program are relatively close. Therefore, the SEISAB transverse displacements are realistic.

5.3.3 The bending moments in the columns, based on the assumptions of the computer model, are:

TRANSVERSE BENDING MOMENTS (kip-ft)

	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	287	171	127	204
Col. 1 - Top	215	129	96	153
Col. 2 - Bottom	346	207	153	246
Col. 2 - Top	341	204	151	243
Col. 3 - Bottom	330	198	147	235
Col. 3 - Top	334	200	148	238
Col. 4 - Bottom	313	187	139	223
Col. 4 - Top	249	158	117	188

LONGITUDINAL BENDING MOMENTS (kip-ft)

JOINT	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	314	526	521	314
Col. 1 - Top	0	425	415	0
Col. 2 - Bottom	314	526	521	314
Col. 2 - Top	0	425	415	0
Col. 3 - Bottom	314	526	521	314
Col. 3 - Top	0	425	415	0
Col. 4 - Bottom	314	526	521	314
Col. 4 - Top	0	425	415	0

Inspection of the SEISAB computer printout revealed that longitudinal forces created very small transverse moments and transverse forces crasted very small longitudinal moments. Therefore, increases due to combinations of longitudinal and transverse forces are insignificant.

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5.3.4 It is estimated that the three-pile bent of the center segment will have transverse displacement 40% greater than those computed for bent 2 of the computer model. Other bent displacements should not exceed those of the computer model by more than 33%.

5.3.5 The interaction diagram for the columns shows that the bending capacity of the columns without axial compression is 175 kip-ft. This condition occurs at the batter piles, where tension forces are offset by the weight of the structure. Moment capacities of columns supporting axial compression are higher.

The columns are spirally reinforced and have good ductility. The force adjustment factor (Z) is 8.0.

There is no confinement reinforcement in the column to bent cap connection areas. It is assumed that there will be loss of flexural capacity in these connections due to loss of bond of the vertical column bar anchorage or shear degradation within the joint. It is further assumed that elastic moments which should be resisted at the top of the column will be redistributed to the point of fixity.

Maximum bending moments in the computer model occur in the longitudinal direction at the top of bent #3 (425 kip-ft). If these moments are entirely redistributed to the point of fixity, the column base moment becomes 526 + 425/2 = 740 ft-kips. This represents a required adjustment factor (Z) of 740/175 = 4.2.

It is estimated that maximum elastic bending moments in some columns of Viaduct #4 will be 40% higher than those indicated by the computer model. Therefore, the required adjustment factor (Z) may be as high as 4.2 x 1.4 = 5.9. This value is still within acceptable limits.

5.3.6 Displacements are acceptable. Six tenths of an inch longitudinal displacement is only 6% of the effective bearing width.

Transverse displacement is slightly restrained by a 2-in.-deep by 3-ft-wide keyway. Some damage to the keyway interface can be expected, but structural stability will not be impaired.

6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION

No structural modification of this viaduct is required.

Related Work:

Check drainage system and perform minor repairs	$2,000
Engineering supervision and inspection	1,000
Total	$3,000

7. SUMMARY AND CONCLUSIONS

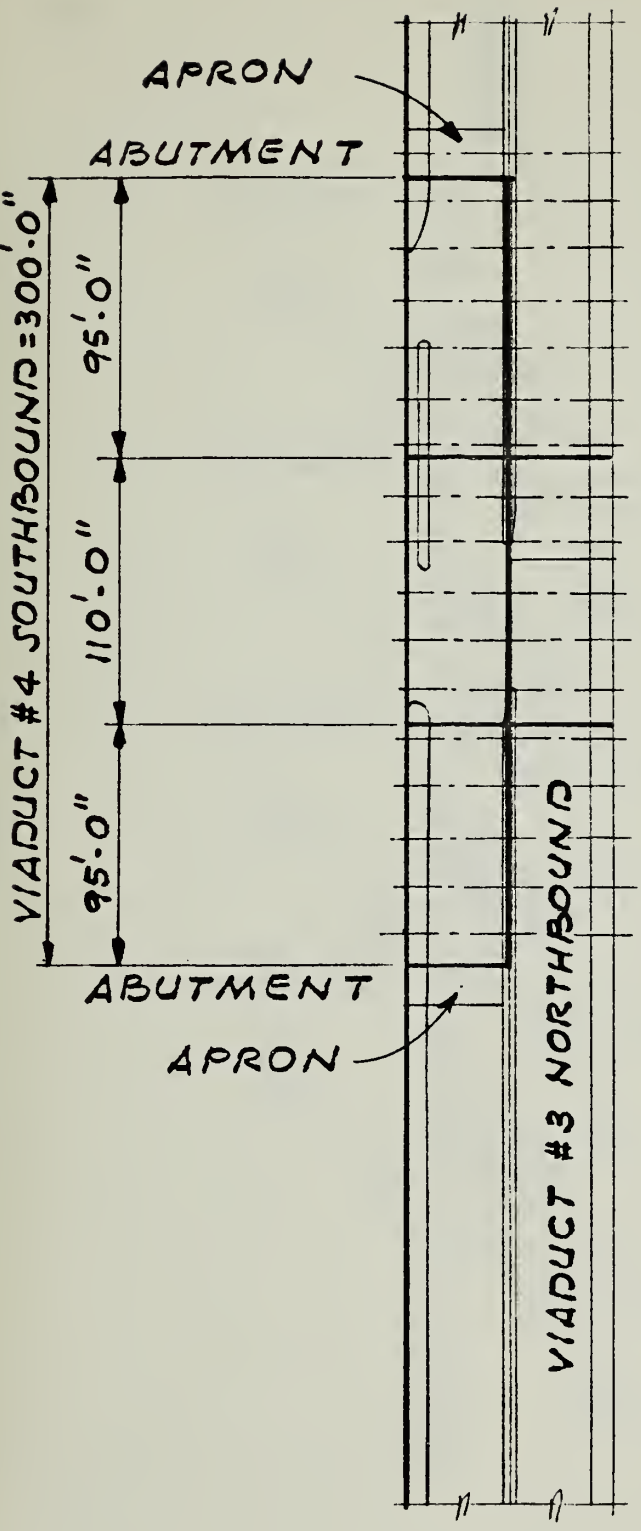
Market Street Sidehill Viaduct #4 supports the southbound lanes of Market Street just north of 23rd Street. It is constructed almost entirely on grade, so that the supporting cast-in-place columns are completely embedded except for the most easterly row.

The peak ground acceleration is 0.7g and the maximum spectral acceleration is 1.6g.

Displacements of the structure under the postulated earthquake forces are acceptable. Inelastic behavior is anticipated and some redistribution of moments may occur in the columns where the concrete is unconfined. Some concrete crushing may occur at the interface of the keyways in the transverse expansion joints. There should be no life safety hazard, and the structure should continue to function.

The conclusions above are dependent upon the soil under the structure remaining stable. Saturation should be avoided.

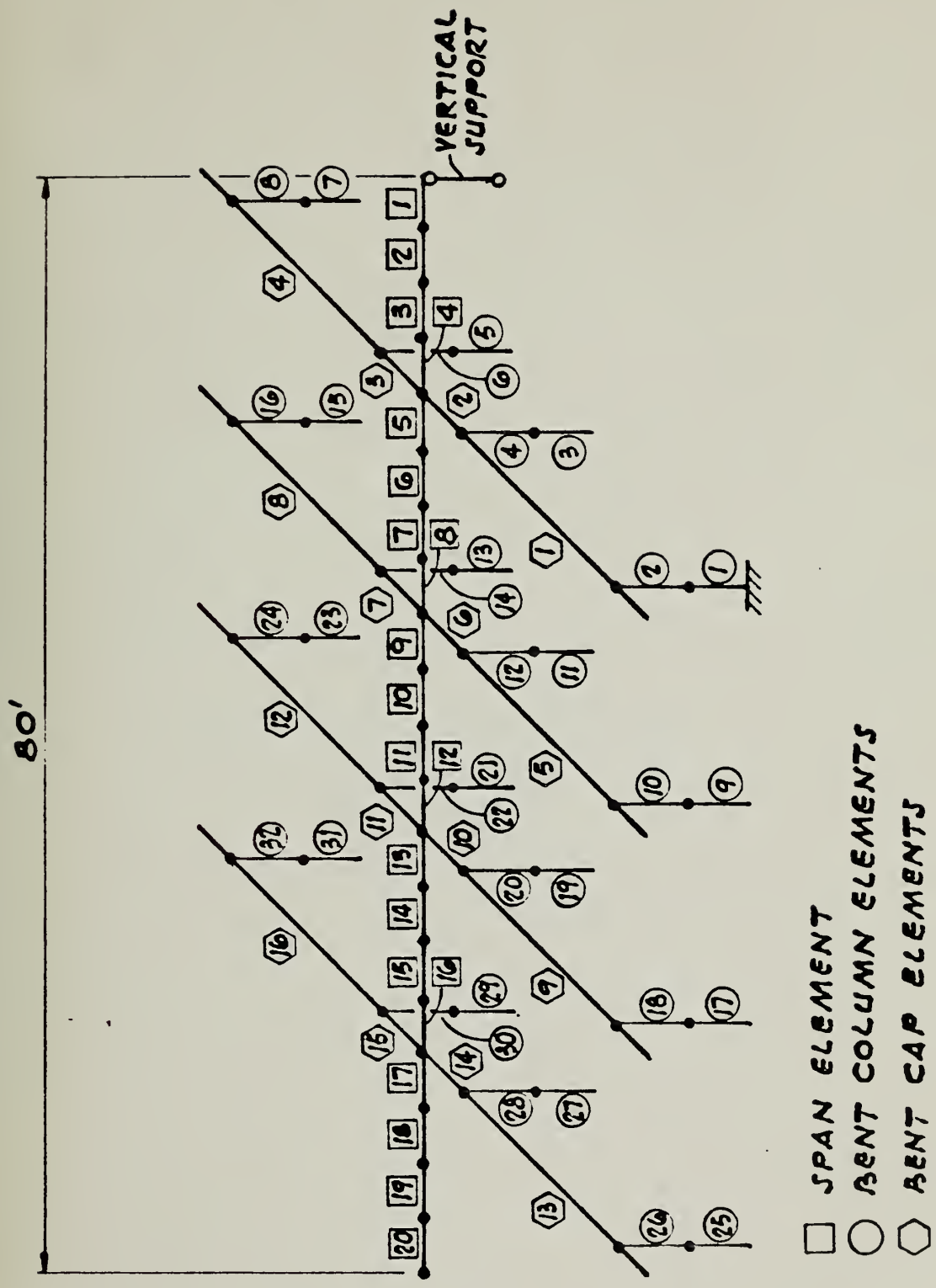
We recommend that periodic maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.



PLAN

PLAN
MARKET STREET VIADUCT # 4
S.F. BRIDGE GROUP II A-4

FIGURE 1



COMPUTER MODEL FOR TYPICAL SEGMENT
MARKET STREET VIADUCT #4
S.F. BRIDGE GROUP II A-4

FIGURE 2

MARKET STREET VIADUCTS #5 & #6

IIA-5 & IIA-6

MARKET STREET SIDEHILL VIADUCTS NOS. 5 & 6

CONTENTS

		page
1.	INTRODUCTION	IIA-5.1
2.	SITE INVESTIGATION	IIA-5.2
3.	BRIDGE DESCRIPTION	IIA-5.4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING BRIDGE	IIA-5.7
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE	IIA-5.8
6.	RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-5.12
7.	SUMMARY AND CONCLUSIONS	IIA-5.13

REFERENCES ..... APPENDIX C

FIGURES:

1.	PLAN	IIA-5.14
2.	TYPICAL BENTS	IIA-5.15
3.	COMPUTER MODEL -VIADUCT #6	IIA-5.16

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1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO] has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

2. SITE INVESTIGATION

Results of the site investigation for all bridges is given in Reference 3. A general summary of the reference as it pertains to the Market Street Sidehill Viaducts Nos. 5 & 6 is given in this section.

The viaducts are parallel, cast-in-place concrete vehicular bridges along Market Street at Corbett Avenue. Viaduct No. 5 supports the northbound lanes, and Viaduct #6 supports the southbound lanes.

Available geological information indicates that the site is underlain by undivided surficial deposits (Qu). The underlying bedrock is Franciscan Formation of the Central Highlands belt. The depth to the bedrock is estimated to be approximately 40 ft below the sloping ground. An exploratory boring, DH-14, was drilled at the intersection of 24th Street and Grand View Avenue to explore the subsurface conditions. The boring was 31.5 ft deep, and the materials encountered consisted of stiff to very stiff, sandy clay (CL) with an approximately 6-ft-thick layer of stiff silty clay (CL) starting at the depth of 12 ft. Standard blow counts of the sandy clay (CL) ranged from 13 to 29 per foot of penetration, and standard blow count of silty clay (CL) was 15 per foot of penetration. Ground water level was encountered at the depth of 24 ft in the boring.

The viaduct is located at approximately 6 miles northeast of the San Andreas fault and 13 miles southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 24 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum II, shown in Appendix B are recommended for the seismic safety evaluation.

The cohesive nature of the soil underlying the viaduct and the low ground water level suggest that liquefaction is not likely. Plate 10 of Reference 3 shows that the viaduct is located in the area of potential landslide. Our reconnaissance of the site, however, did not indicate any signs of land instability. No slides were identified at the site by available published data. Geological maps at the Department of Public Works, City and County

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of San Francisco, indicate, however, that several small slides previously took place at the vicinity of the site. Two of the slides were located close to the viaduct between Elizabeth Street and 24th Street and between 24th Street and 25th Street on Grand View Avenue. These slides ocurred in 1941, and the existing records indicate that the slides were repaired in 1941 and 1942. We believe that the risk of landslide during earthquake is not significant. Minor downhill ground movement, however, may be induced by a major earthquake. Such movement may result in minor structural cracking. We recommend that periodical maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.

The site is located at a great distance from the areas of potential lateral spreading and tsunamni inundation as shown on Plates 3 and 11 of Reference 3.

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3. DESCRIPTION OF BRIDGE

3.1 Location:

The Market Street Sidehill Viaducts Nos. 5 and 6 are located in the Twin Peaks portion of Market Street at Corbett Avenue. The viaducts are parallel and contiguous over most of their length. Viaduct No. 5 supports the northbound lanes of Market Street, whereas Viaduct No. 6 supports the southbound lanes.

3.2 Physical Description:

Viaduct No. 5 is 210 ft long and includes two 15-ft end spans and nine 20-ft intermediate spans. Viaduct No. 6 is 170 ft long and includes two 15-ft end spans and seven 20-ft intermediate spans. Each viaduct is divided into two segments by an expansion joint, located 5 ft from a supporting bent. The structures are 42 ft wide and each supports an 8-ft wide sidewalk, a 32-ft roadway, and half of a 4-ft median.

Bents are supported on 2-ft-diameter, cast-in-place concrete piles. The piles are spirally reinforced. In the following schedule, these pile bents have three vertical piles. At the four pile bents the two exterior piles are vertical while the two interior piles are on a 1:4 batter.

Station	No. of Piles	
	Viaduct #5	Viaduct #6
47 + 40 - Abutment, Via. #5	3	
+ 55	3	
+ 75	4	
+ 95	3	
48 + 00 - Abutment, Via. #6		3
+ 15	4	3
+ 20 - Exp. Jt., Via. #5		
+ 35	3	4
+ 55	4	3
+ 75	3	4
+ 95	3	3
49 + 00 - Exp. Jt., Via. #6		
+ 10 - Abutment, Via. #5	4	
+ 15		3
+ 35		4
+ 55		3
+ 75		4
+ 95		3
45 + 10 - Abutment, Via. #6		4

The deck between bents is a 12-in.-thick reinforced concrete slab. Viaduct No. 5 is divided by the expansion joint into one 100-ft-long segment and one 110-ft-long segment. The segment lengths on Viaduct No. 6 are 80 ft and 90 ft.

Most of the structures were constructed directly on grade and virtually none of the underside of the roadway is visible.

The viaducts were constructed in 1957 as part of a general widening and realignment of Market Street between Hattie Street and Corbett Avenue.

3.3 Traffic Flow:

Market Street provides the most direct route between the financial and commercial center of San Francisco and the western residential districts between Golden Gate Park and Lake Merced and therefore carries heavy passenger and truck traffic. A detour to the north of Twin Peaks, if required by failure of any structure on Market Street, would add about 1.6 miles to the distance across the City.

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3.4 Description of Bridge Condition:

Very little of the underside of this viaduct or its supporting columns are visible for inspection. What could be seen is in good condition.

3.5 Description of Specialty Items:

The deck and bent caps are monolithic and the bent caps are cast directly on the supporting piles. The only part subject to movements are the expansion joints which are located at the quarter points in the deck slabs and at the abutments.

Expansion joints are constructed with a 12-in.-wide bearing surface across the full width of the deck. Contact surfaces are separated by two 1/16-ft thick layers of asbestos sheet packing.

Expansion joints are in good condition.

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4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING BRIDGE

4.1 The criteria for analysis of the existing bridge, as well as for the modified bridges, are given in Appendix A. A general summary of the appendix as it pertains to the Market Street Sidehill Viaducts Nos. 5 and 6 is given below:

4.1.1 The soil investigation shows that the bridge is situated on alluvium with 40-ft depth to bedrock. The Caltrans Response Spectrum II, shown in Appendix B, was used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the peak ground acceleration (PGA) is 0.7g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where
- U = Total load to be resisted
- $D.L.$ = Dead load
- $E.Q.X$ = Earthquake load in X-direction (longitudinal)
- $E.Q.Z$ = Earthquake load in Z-direction (transverse)
- Z = Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Multicolumn pier (spirally reinforced)	8
Column pier connections	2

4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete f'_c	= 2,500 psi
Reinforcing Steel f_y	= 40,000 psi*

*The steel grade is not specified in the plans. However, based on the 1950 construction date, it is reasonable to assume that A15 Grade 40 steel was used.

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5. SEISMIC ANALYSIS OF EXISTING BRIDGE

5.1 Analysis Method:

Market Street Viaducts Nos. 5 and 6 were analyzed for seismic forces using the computer program SEISAB, Version 1.3, written by the Engineering Computer Corporation. The program does not make provisions for battered columns, so the following procedures were used for the bents with battered piles:

- An equivalent frame was assumed for the bents. This frame used vertical piles in lieu of battered piles, but the moment of inertia was tripled. The equivalent frame was used in the SEISAB program.
- The actual bent configuration was modeled for the FRAME2C computer program. The stiffness of this frame was computed and compared with the stiffness of the vertical frame, and member moments and shears were determined for a 100-kip lateral force.
- Total lateral force per bent was computed from the SEISAB model and then the FRAME2C results were factored up or down as required.

5.2 Modeling Assumptions:

5.2.1 The Market Street viaduct are all constructed with typical segments from 80 to 110 ft long. There is no longitudinal tension restraint between segments. There is a very small (2 in. deep) keyway providing transverse restraint. It has been neglected.

5.2.2 The computer model was based on the south segment of Viaduct No. 6. It is assumed to be a five-span continuous concrete structure. The 15-ft end spans are vertically supported but unrestrained in the longitudinal and transverse directions by abutments. One abutment is real, the other is fictitious. Three of the four bents have three vertical piles in each bent. One bent, adjacent to the midspan, has battered piles.

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5.2.3 The ground surface under most of Viaducts Nos. 5 and 6 is directly under the slab, and the columns are not exposed. For the computer model, however, it was assumed that the cast-in-place concrete piles are unrestrained to a depth of 6 ft.

5.3 Results of Analysis:

5.3.1 The predominate periods of vibration in the two directions, along with their corresponding spectral acceleration (S_a) and participation factors, are given below:

	T (sec)	S_a (g)	Participation Factor	Mode
X-Direction (Longitudinal)	0.10	1.28	4.72	1
Z-Direction	0.07	1.16	2.09	2
(Transverse)	0.06	1.10	4.50	3

5.3.2 The deflections of the structure, based on the assumptions of the computer model are:

Longitudinal (S_x) 0.11 in.

Transverse (S_z)

Abut. #1	0.06
Bent #2	0.05
Bent #3	0.03
Bent #4	0.03
Bent #5	0.04
Abut. #6	0.06

The transverse stiffnesses of the bents as modeled from the SEISAB computer program and as actually computed by the FRAME2C program are relatively close. Therefore, the SEISAB transverse displacements are realistic.

5.3.3 The bending moments in the columns, based on the assumptions of the computer model, are:

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TRANSVERSE BENDING MOMENTS (kip-ft)

	BENT 2	BENT 3	BENT 4	BENT 5
Col. 1 - Bottom	132	107	91	117
Col. 1 - Top	85	79	60	77
Col. 2 - Bottom	186		124	159
Col. 2 - Top	162		106	135
Col. 2a - Bottom		315		
Col. 2a - Top		223		
Col. 2b - Bottom		315		
Col. 2b - Top		223		
Col. 3 - Bottom	132	107	91	117
Col. 3 - Top	85	79	60	77

LONGITUDINAL BENDING MOMENTS (kip-ft)

JOINT	BENT 2	BENT 3	BENT	4BENT 5
Col. 1 - Bottom	273	221	246	269
Col. 1 - Top	0	97	130	0
Col. 2 - Bottom	257		227	260
Col. 2 - Top	0		111	0
Col. 2a - Bottom		212		
Col. 2a - Top		82		
Col. 2b - Bottom		212		
Col. 2b - Top		82		
Col. 3 - Bottom	273	221	246	269
Col. 3 - Top	0	97	130	0

Inspection of the SEISAB computer printout revealed that longitudinal forces created very small transverse moments and transverse forces created very small longitudinal moments. Therefore, increases due to combinations of longitudinal and transverse forces are insignificant.

5.3.4 The interaction diagram for the columns shows that the bending capacity of the columns without axial compression is 175 kip-ft. This condition occurs at the batter piles, where tension

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forces are offset by the weight of the structure. Moment capacities of columns supporting axial compression are higher.

The columns are spirally reinforced and have good ductility. The force adjustment factor (Z) is 8.0.

There is no confinement reinforcement in the column to bent cap connection areas. It is assumed that there will be loss of flexural capacity in these connections due to loss of bond of the vertical column bar anchorage or shear degradation within the joint. It is further assumed that elastic moments which should be resisted at the top of the column will be redistributed to the point of fixity.

Maximum bending moments occur in the longitudinal direction at the top of bent #4 (130 kip-ft). If these moments are entirely redistributed to the point of fixity, the column base moment becomes 246 + 130/2 = 311 ft-kips. This represents a required adjustment factor (Z) of 311/175 = 1.8.

5.3.5 Displacements are negligible. Eleven-hundredths of an inch longitudinal displacement is only 19% of the effective bearing width.

Transverse displacement is slightly restrained by a 2-in.-deep by 3-ft-wide keyway, and the 0.06-in. displacement can be accommodated by the expansion joint material in the keyway.

6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION

No structural modification of this viaduct is required.

Related Work:

Check drainage system and perform minor repairs	$4,000
Engineering supervision and inspection	2,000
Total	$6,000

7. SUMMARY AND CONCLUSIONS

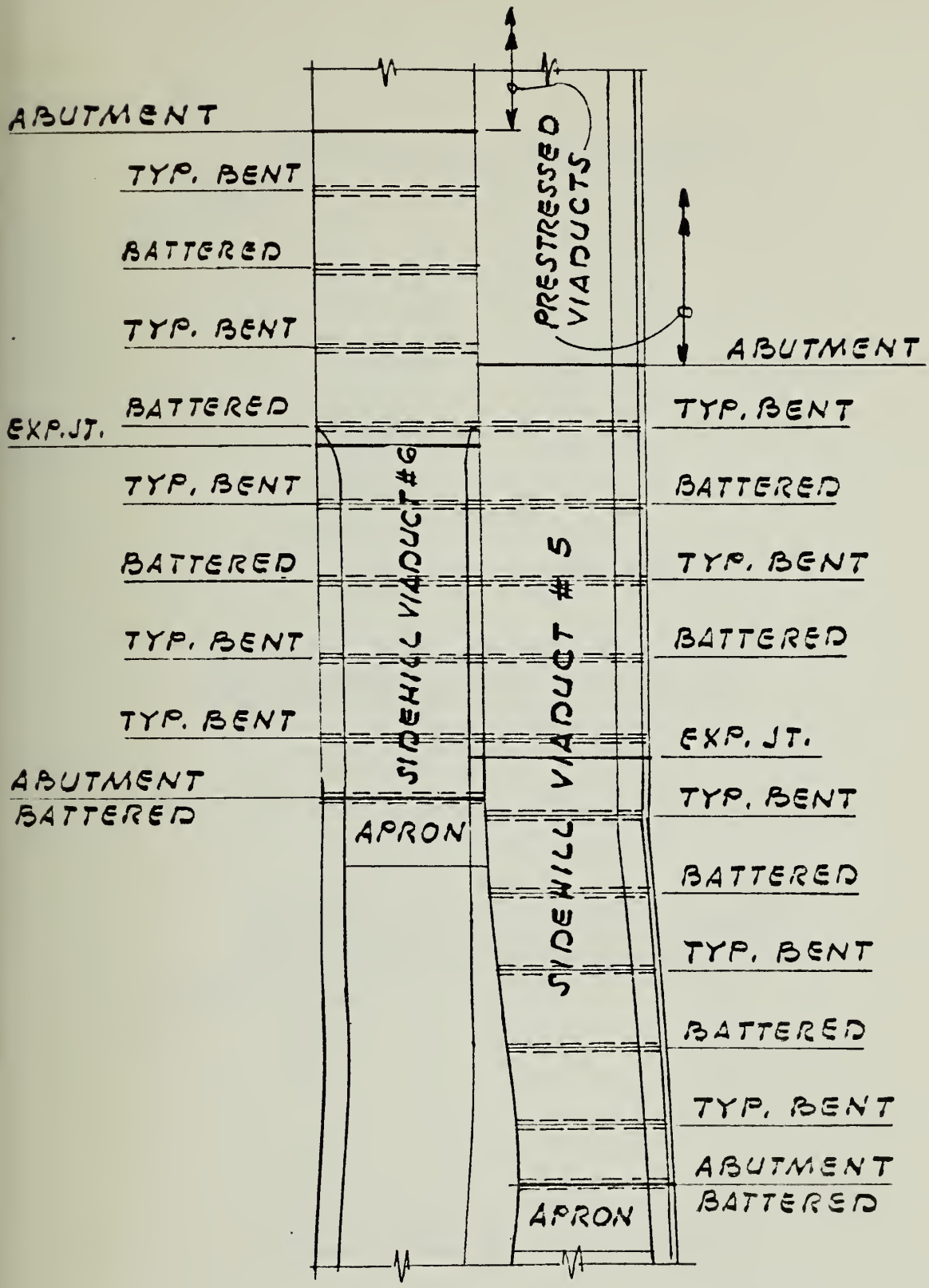
Market Street Sidehill Viaducts Nos. 5 and 6 are parallel structures along Market Street at Corbett Avenue. Viaduct No. 5 supports the northbound lanes, and Viaduct No. 6 supports the southbound lanes. They are constructed almost entirely on grade, so that the supporting cast-in-place columns are completely embedded.

The peak ground acceleration is 0.7g and the maximum spectral acceleration is 1.6g.

Displacements of the structure under the postulated earthquake forces are acceptable. Inelastic behavior is anticipated and some redistribution of moments may occur in the columns where the concrete is unconfined. There should be no life safety hazard, and the structure should continue to function.

The conclusions above are dependent upon the soil under the structure remaining stable. Saturation should be avoided.

We recommend that periodic maintenance be provided for all drainage systems. Flooding, ponding, and saturation of the soil, which would cause significant reduction of soil strength, should be avoided. All damage caused by erosion should be repaired immediately.

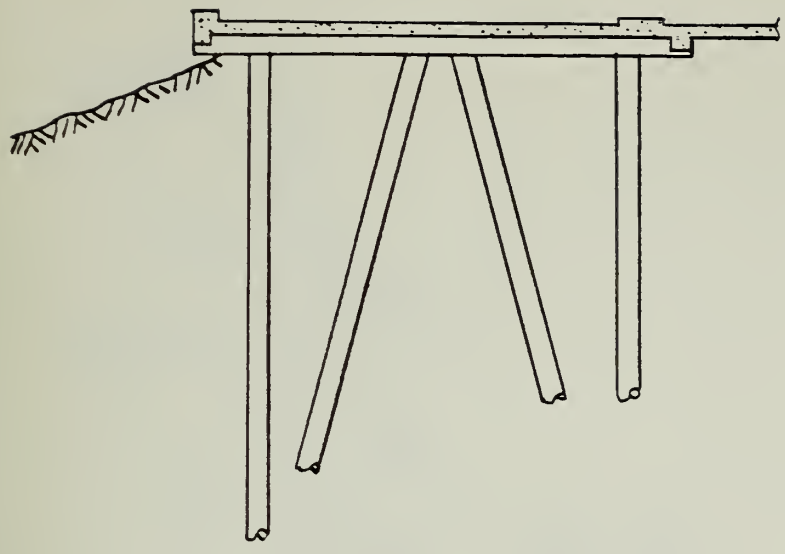


PLAN

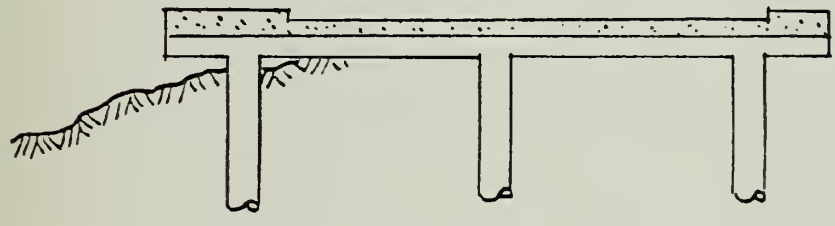
N

PLAN
MARKET STREET VIADUCTS # 5 & 6
S.F. BRIDGE GROUP IIA-5 & 6

FIGURE 1



BATTERED PILE BENT



TYPICAL BENT

ENT TYPES
MARKET STREET VIADUCTS # 5 & 6
F. BRIDGE GROUP IIA-5 & 6

FIGURE 2

COMPUTER MODEL FOR TYPICAL SEGMENT
MARKET STREET VIADUCT #6
S.F. BRIDGE GROUP IIA-5 & 6

FIGURE 3

HARRISON STREET VIADUCT OVER BEALE STREET

IIA-7

HARRISON STREET VIADUCT OVER BEALE STREET

CONTENTS

		page
1.	INTRODUCTION	IIA-7.1
2.	SITE INVESTIGATION	IIA-7.2
3.	BRIDGE DESCRIPTION	IIA-7.3
4.	CRITERIA FOR ANALYSIS OF THE EXISTING BRIDGE	IIA-7.5
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE AND MODIFIED BRIDGE	IIA-7.7
6.	RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-7.8
7.	SUMMARY AND CONCLUSIONS	IIA-7.10
REFERENCES		APPENDIX C

FIGURES:

1.	LONGITUDINAL AND TRANSVERSE SECTIONS	IIA-7.11
2.	RECOMMENDED REMEDIAL WORK	IIA-7.12
3.	COMPUTER MODEL	IIA-7.13

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1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under the Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards, and if remedial work is required, to recommend a remedial program and estimate the cost.

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2. SITE INVESTIGATION

Results of the site investigation for all nine bridges are contained in Reference 3. A summary of the reference as it pertains to the Harrison Street Viaduct over Beale Street is given below.

Available geological information indicates that the viaduct lies in a bedrock outcrop area. An exploratory boring, DH-8, was drilled in Harrison Street at approximately 50 feet to the east of the viaduct to explore the subsurface conditions. The boring was 10 feet deep and the bedrock was encountered beneath some 12 inches of asphalt and concrete pavement. The bedrock is sandstone of Franciscan Formation of the Nob Hill belt.

The viaduct is located at approximately 9.5 mi northeast of the San Andreas fault and 9.5 mi southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 12 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum I, shown in Appendix B are recommended for the seismic safety evaluation.

The viaduct is located on the rock outcrop and the risk associated with liquefaction, landsliding, and lateral spreading is minimal. Some damage caused by erosion was noticed near the viaduct abutments. Such damage should be repaired. It may be preferable to gunite a portion of the abutment slopes. Plate 11 of Reference 3 shows that the viaduct is located at great distance from the areas of potential tsunami inundation.

3. DESCRIPTION OF BRIDGE

3.1 Location:

The Harrison Street Viaduct carries Harrison Street over Beale Street.

3.2 Physical Description:

The viaduct is a three-span continuous girder bridge supporting a 66-ft-6-in. wide roadway and two 8-ft-0-in. wide sidewalks. The center span is 82-ft-6-in. in length, and the east and west spans are 45-ft-0-in. and 53-ft-0-in., respectively. Internal span support points are provided by two abutment walls, both of which were salvaged from a pre-existing steel bridge dating to approximately 1910. The east abutment wall extends approximately 21' above the street level (measured to the bottom of main girders) and the superstructure has been cast solidly into this existing wall providing a connection which is fixed with respect to all translational degrees of freedom. The west abutment wall extends approximately 31' above street level and vertical support is provided with bronze bearing plates at the interface between this wall and each of the five main girders. Lateral support is provided at this abutment by two shear keys bearing through 1/2-in. lead plates. Thus, the west abutment wall connection is designed to provide a minimum of restraint for motion in the longitudinal direction. At each of the far east and west ends of the bridge, abutments are provided supporting the end spans. Each is embedded a minimum of 1-ft-6-in. into bedrock, and vertical support is provided through bronze bearing plates which are keyed to allow longitudinal motion only. The bottom edges of the center three girders are provided with 1/2"x16"x16" lead bearing plates furnishing transverse restraint.

3.3 Traffic Flow:

Traffic counts on both Harrison and Beal Streets have not been obtained. However, observed traffic is light and alternate routes are available for both streets.

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3.4 Description of Bridge Condition:

The concrete abutments, wing walls and superstructure all appear to be in good condition. The erosion noted in the soil investigation appears to be limited to soil above rock. Further erosion is not likely to occur, although periodic inspections are recommended.

3.5 Description of Specialty Items:

The west abutment wall supporting the superstructure has bronze bearing plates, details for which are not available. Due to the height above the street at which these bearings are located, we were not able to visually inspect these items. However, the drawings indicate that a 3" open joint should be present along the plane of these bearings and therefore, with proper equipment, visual inspection should be quite straightforward if deemed necessary at some future date.

The far east and west abutments have not been provided with access from underneath the superstructure, therefore, inspection of bearings at these locations is not possible.

4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE

4.1 The criteria for analysis of the existing bridge, as well as for the modified bridges, are given in Appendix A. A general summary of the appendix as it pertains to the Harrison Street Viaduct over Beale Street is given below:

4.1.1 The soil investigation shows that the bridge is situated on bedrock. The Caltrans Response Spectrum I, shown in Appendix B, is used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the maximum spectral acceleration is 1.3g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where

U = Total load to be resisted

$D.L.$ = Dead load

$E.Q.X$ = Earthquake load in X-direction (longitudinal)

$E.Q.Z$ = Earthquake load in Z-direction (transverse)

Z = Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Component	Z
Wall-type pier	2*
Abutment connection	0.8

The Z factor is to be used for modifying the bending moment of the columns. For shear or axial force, a Z-factor of 1.0 shall be used.

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4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete f'_c = 2,500 psi

Reinforcing Steel f_y = 40,000 psi

5. SEISMIC ANALYSIS OF EXISTING AND MODIFIED BRIDGE

5.1 Assumptions:

5.1.1 The superstructure and abutment walls were modeled using uncracked section properties. This is believed to furnish sufficiently accurate results providing a high degree of ductility is not required. Longitudinal support was assumed to be provided only by the east supporting wall, as the remainder of the vertical supports are on brass bearing plates and keyways are only provided for transverse loading. The structure was modeled as transversely supported on both end abutments and both supporting walls.

5.1.2 The analyses was made using the SEISAB computer program. (Reference 5).

5.2 Results of Analysis:

Initial computer runs were made using response spectrum Type II* (10' to 80' alluvium) even though the bridge is founded on bedrock (0' to 10' alluvium, Type I Spectrum). Even for this significantly higher spectral input, the superstructure was found to respond within the acceptance criteria using Z=1.0 in almost all cases (i.e., elastic response). It was determined inadequate shear capacity was available in the transverse direction. Therefore the structure was reanalyzed using Type I spectra to obtain demand shears corresponding to the 0' to 10' alluvium existing at this site. Transverse shear capacity at the end abutments was still found to be exceeded. It was found possible to remedy this exceedance by either if two retrofit schemes which are discussed in the recommendations section of this report.

*Spectrum types refer to Figure 3a of Reference 3.

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6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL REHABILITATION

6.1 Recommendations - First Scheme:

The first scheme for retrofitting the structure consists of increasing the transverse shear capacity at the end abutments by addition of two shear panels in the outside bays at each end abutment. Such shear panels would be doweled into the abutment to provide the necessary shear capacity, and the sides cast against teflon bearings to provide restraint against transverse motion while still allowing the existing bronze bearings to function as a vertical support and permitting longitudinal motions to occur per the original design.

The first scheme has a major disadvantage in that physical access to this region is extremely limited, and may be obtained either through existing 2'x3' opening in the interior abutment walls (currently sealed with steel grating), or by demolition or reinforced concrete walls parrellel with the centerline of the bridge. Additionally, headspace is limited in this area below the superstructure.

6.2 Recommendations - Second Scheme:

The second scheme proposed for retrofitting this structure requires the supporting walls take the entire transverse force. The east supporting wall was built with the superstructure and the existing wall cast together so it is equivalent to monolithic construction. It has the capacity to resist the additional shear. The superstructure over the west supporting wall is supported on bronze bearing. Transverse shear resistance is provided by two shear keys. One-half-inch thick lead plates on the vertical surface are used to permit movement. The outer portions of the wall are separated from the superstructure by a 1/2-in. vertical joint. The retrofit consists of placing a steel and teflon bearing pad in this vertical joint so the outer portion of the wall will contribute to the shear capacity.

Scheme 2 will not prevent transverse displacement of the bridge girders at the end abutments. However, the east and west supporting walls will limit transverse displacement to the extent that damage to the

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bearing plates will be nominal and the bridge should continue to be usable. Inspection of the bearings is advisable, however, after a major seismic event.

6.3 Cost Estimate - Scheme 2:

Description	Quantity	Unit	Unit Price	Total
Mobilization		L.S.		$1,000
Clean and Prepare Joints		L.S.		1,500
Furnish and Install Bearings	2	ea.	500.00	1,000
	Subtotal			$3,500
	Contractors OH & P (20%)			700
	Contingency (20%)			840
	Total			$5,020
	Engineering @ 20%			1,000
	Escalation @ 8%			400
			TOTAL	$6,420

Related Erosion Control Work - Optional:

Gunite embankment at abutments	$15,000
Engineering supervision and inspection	3,000
Total	$18,000

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7. SUMMARY AND CONCLUSIONS

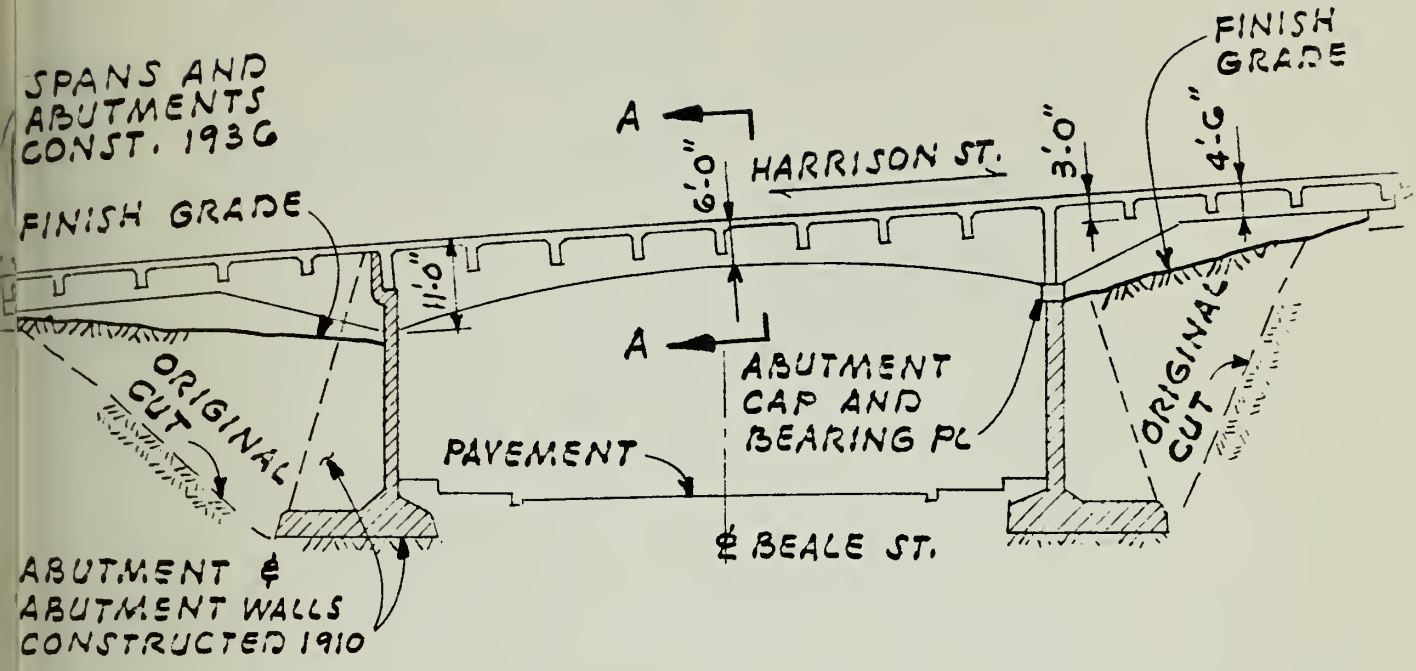
The Harrison Street Viaduct is a three-span structure carrying Harrison Street over Beale Street. The superstructure is beam and slab construction supported by five primary girders.

The peak rock acceleration is 0.5g and the maximum spectral acceleration is 1.3g.

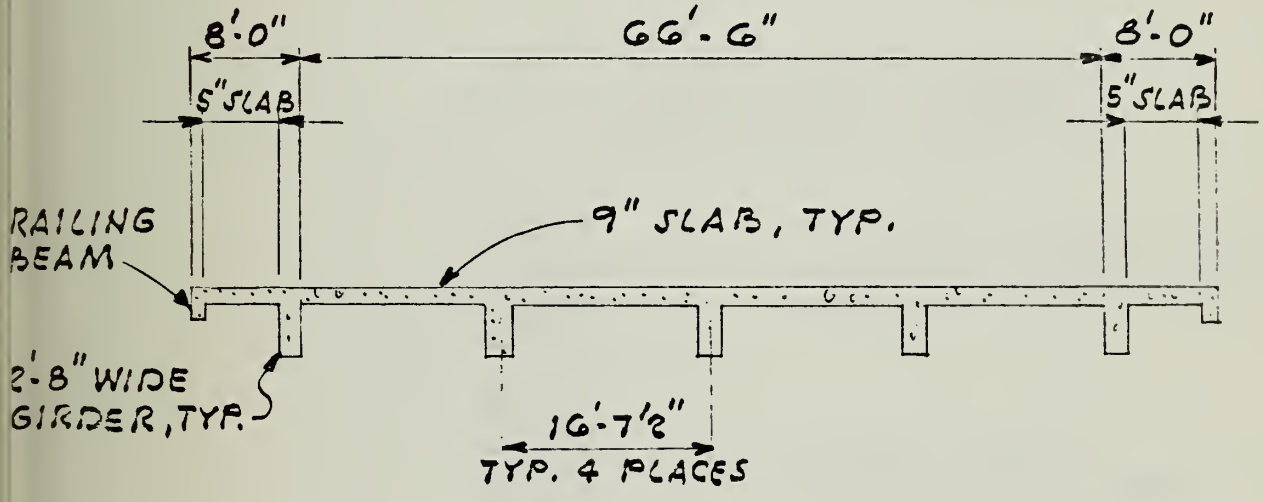
Damage to this structure would not be catastrophic and alternate routes are available. The remedial work proposed may be given low priority.

Retrofit measures are recommended to increase the transverse shear capacity of the west supporting wall. It is possible that transverse displacement during a major seismic event will damage the keeper lugs of the end abutment bearing plates, but the bridge should remain serviceable. The end abutment bearings should be inspected after a major event.

The ground under the side wing walls should be inspected periodically for evidence of increasing scour. No rework of this area is recommended at this time.

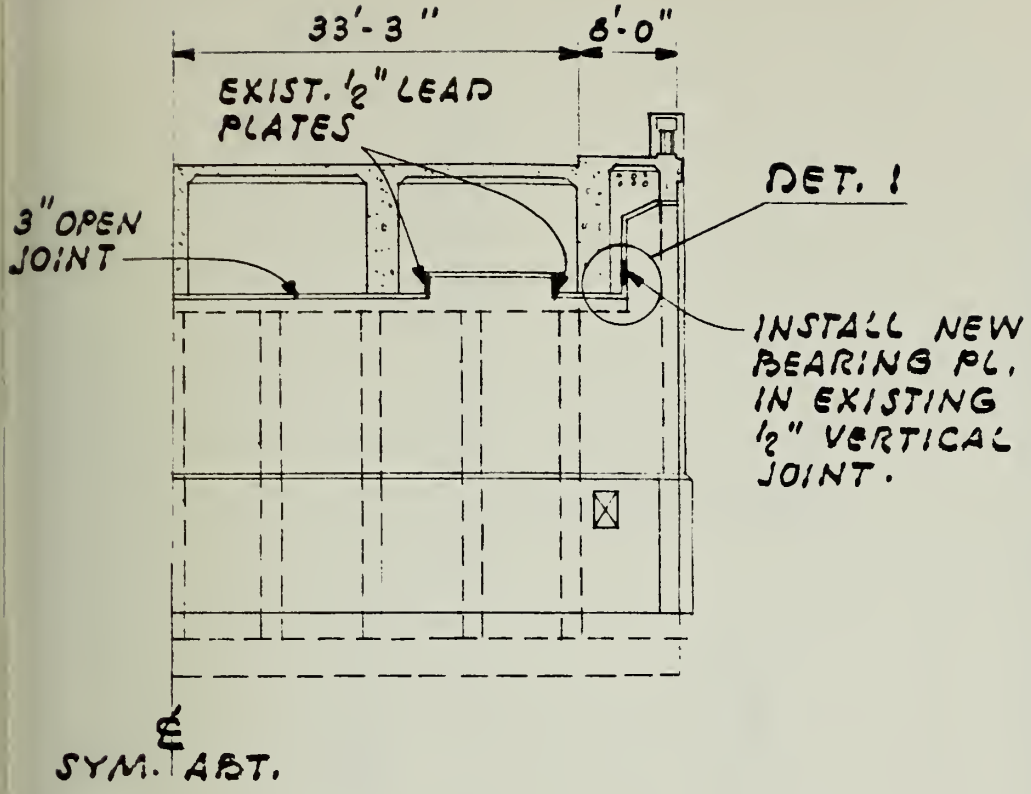


LONGITUDINAL SECTION

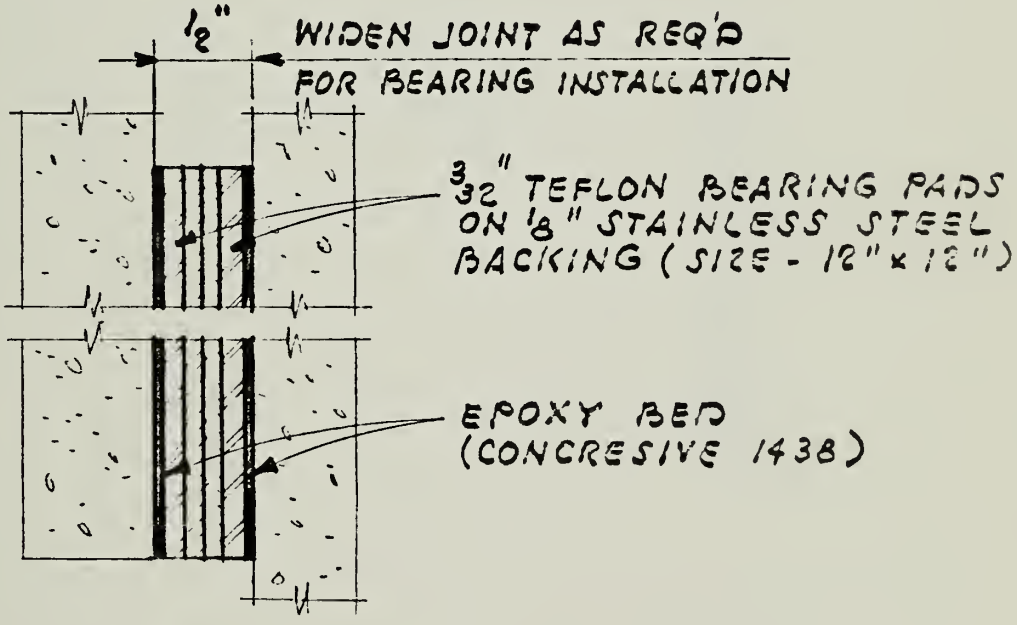


TRANSVERSE SECTION A-A

LONGITUDINAL AND TRANSVERSE SECTIONS
HARRISON ST. VIADUCT OVER BEALE ST.
S.F. BRIDGE GROUP II A-7 FIGURE 1



WEST SUPPORTING WALL


½"
WIDEN JOINT AS REQ'D FOR BEARING INSTALLATION
3/32" TEFLON BEARING PADS ON ⅛" STAINLESS STEEL BACKING (SIZE - 12" x 12")
EPOXY BED (CONCRESIVE 1438)

DETAIL 1

ECOMMENDED REMEDIAL WORK

ARRISON ST. VIADUCT OVER BEALE ST.

F. BRIDGE GROUP IIA-7

FIGURE 2

PLANAR MODEL WITH 3 DIMENSIONAL PROPERTIES

COMPUTER MODEL
HARRISON ST. VIADUCT OVER BEALE ST.
SF. BRIDGE GROUP II A-7

FIGURE 3

BRYANT STREET VIADUCT BETWEEN 2ND AND BEALE

IIA-8

BRYANT STREET VIADUCT BETWEEN SECOND AND BEALE STREETS

CONTENTS

		page
1.	INTRODUCTION	IIA-8.1
2.	SITE INVESTIGATION	IIA-8.2
3.	BRIDGE DESCRIPTION	IIA-8.4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE	IIA-8.6
5.	SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE	IIA-8.7
6.	RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-8.9
7.	SUMMARY AND CONCLUSIONS	IIA-8.10

REFERENCES ..... APPENDIX C

FIGURES:

1.	PLAN	IIA-8.13
2.	ELEVATION AND SECTION - PROPOSED WALLS	IIA-8.14
3.	SECTIONS MODIFIED HINGE	IIA-8.15
4.	COLUMN ENCASEMENT ALTERNATE TO SHEAR WALLS	IIA-8.16
5.	COMPUTER MODEL	IIA-8.17

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1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

2. SITE INVESTIGATION

Results of the site investigation for all bridges is given in Reference 3. A general summary of the reference as it pertains to the Bryant Street Viaduct Between Second and Beale Streets is given in this section.

This viaduct is a cast-in-place concrete vehicular bridge along Bryant Street between Second and Beale Streets.

Available geological information indicates that the site is underlain by sandstone and shale of Franciscan Formation of the Nob Hill belt.

The viaduct is located at approximately 9 miles northeast of the San Andreas fault and 10 miles southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 12 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum I, shown in Appendix C are recommended for the seismic safety evaluation.

The viaduct is founded on rock outcrop and the risk associated with liquefaction, landsliding, and lateral spreading is minimal. The site is located in the area of tsunami inundation as shown on Plate 11 of Reference 3.

Additional earth pressure may be induced by the earthquake on the abutments. The seismically induced earth pressure is estimated to be equivalent to a uniform pressure of 20 pcf times the wall height in feet. The seismically induced earth pressure should be added to active and at-rest pressures. The active and at-rest pressures are estimated to be equivalent to the pressures exerted by a fluid weighing 35 and 55 pcf, respectively. Lateral pressures induced by appropriate surcharge should also be taken into account. The passive pressure to be mobilized against buried walls for the resistance of lateral forces may be taken as the pressure exerted by a fluid weighing 300 pcf. When combining passive pressure and friction

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to resist lateral pressure, the passive pressure component should be reduced by one-third. Resistance to lateral forces may be provided by friction between concrete foundations and the underlying subgrade. A friction factor of 0.45 may be used for this purpose. The above pressures assume proper drainage behind the wall without water pressure. All existing drains and weepholes should be checked and cleaned if required. Additional weepholes are recommended if the existing weepholes are spaced greater than 8 feet apart. All weepholes should have a diameter of at least 6 inches and should extend through the stem of abutments and should be filled with drain materials and protected with filter cloth. The drain materials should be extended at least 1 foot beyond the inner face of the wall and should meet the requirements for Class 1, Type B, Permeable Material, Section 68, Caltrans Standard Specifications.

The backfill materials behind the abutments may be densified due to the earthquake shaking. Settlement up to several inches may be caused by a major earthquake.

A bearing pressure of 5,000 psf may be used for the footings. The pressure may be increased by one-third for seismic and other transient loading conditions.

3. DESCRIPTION OF BRIDGE

3.1 Location:

The Bryant Street Viaduct is located between Second and Beale Streets.

3.2 Physical Description:

The viaduct is made up of two very distinct regions structurally referred to in the following discussion as the Northern and Southern Ends.

The Southern End of the viaduct is an elevated bridge supported by 4-ft-square columns with spread footings. Columns are spaced at 55 ft, 3-1/2 in. longitudinally and at 42 ft, 6 in. transversely. The structural deck is composed of parabolic longitudinal girders supporting transverse beams spaced at approximately 11 ft on center. A structural slab spans between the transverse beams. This "southern portion" of the bridge has seven spans for a total length of 387 feet. Spanning from the south end, 1 hinge is provided in the first span, 2 in the third span, 2 in the fifth span, and 1 in the seventh span for a total of six hinges. One hinge of each pair of hinges within a single span was designed to allow for longitudinal expansion while the other is fixed. A small keyway provides nominal restraint against transverse displacement at the "free" end hinge. The keyway and #9 dowels combine to provide transverse restraint at the fixed end.

The north end of the viaduct consists of five modules, each being 55 ft, 3-1/2 in. long for a total length of 276 ft, 6 in. The roadway along this portion of the viaduct is placed directly upon fill material which is placed between retaining walls.

3.3 Traffic Flow:

This portion of Bryant Street carries two-way traffic. The southbound lane serves solely as access to the Bay Bridge on-ramp at Sterling Street. Traffic on this portion of Bryant Street is relatively light and adequate detour routes are available should closure of this section or roadway be required.

3.4 Description of Bridge Condition:

This bridge was observed to be in good condition.

3.5 Description of Specialty Items:

Bearings supplied for supporting the longitudinal parabolic girders are ordinary 1/2-mild steel plates with #4 bars welded to them providing anchorage. Sizes provided are either 1/2x15x26 or 1/2x15x34. In bearings denoted as being fixed-bearing plates, the top and bottom steel plates bear directly upon one another and three #9 bars, 8'-0" long are placed through the hinge providing some continuity for longitudinal axial loads. The top and bottom steel plates are separated by 1/4" of asbestos sheet in bearings noted as sliding bearing plates with no provisions for axial continuity. A small shear key has been provided on the inbound side of all bearings to prevent transverse displacements from occurring between adjacent span elements.

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4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE

4.1 The criteria for analysis of the existing bridge is given in Appendix A. A general summary of the appendix as it pertains to the Bryant Street Viaduct between Second and Beale Streets is given below:

4.1.1 The soil investigation shows that the bridge is situated on rock outcrop. The Caltrans Response Spectrum I, shown in Appendix B, was used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the peak ground acceleration (PGA) is 0.6g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where

U	=	Total load to be resisted
D.L.	=	Dead load
E.Q.X	=	Earthquake load in X-direction (longitudinal)
E.Q.Z	=	Earthquake load in Z-direction (transverse)
Z	=	Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Multicolumn bent (tied columns)	3
Column pier connections	1
Shear forces in columns	1

4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete f'_c = 2,500 psi

Reinforcing Steel f_y = 40,000 psi*

*The steel grade is not specified in the plans. However, based on the 1950 construction date, it is reasonable to assume that A15 Grade 40 steel was used.

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5. SEISMIC ANALYSIS OF EXISTING AND MODIFIED BRIDGE

5.1 Analysis Method:

The Bryant Street Viaduct between Second and Beale Streets was analyzed for seismic forces using the computer program SEISAB (Reference 5).

5.2 Modeling Assumptions:

The actual structure possesses a slight skew which was deemed to be not significant from a structural standpoint and was neglected in the analysis.

All sections were modeled based upon uncracked section properties which a most reasonable assumption for the superstructure due to its very large in-plane strength and stiffness, but of questionable validity for the 4'-square columns due to their lacking adequate shear reinforcement and the magnitude of calculated demand loading. This topic will be addressed in more detail in subsequent portions of this report.

The entire elevated portion of this viaduct was modeled with the bents at the North and South Ends modeled as abutments rather than columns. This modeling limitation is a function of the computer program used to perform the analysis and is believed to yeild results with an acceptable degree of accuracy.

5.3 Results of Analysis:

Due to the arrangement of internal hinges in this structure, a number of significant periods in each direction are obtained. These periods are not necessarily closely spaced due to the variation of column heights and thus lateral stiffnesses of adjacent portions of the structure. Significant longitudinal periods are seen to vary from 0.71 sec to 0.40 sec, vertical periods from 0.22 sec to 0.20 sec, and transverse periods from 0.59 sec to 0.24 sec. Two-thirds of these significant periods occur close enough to the peak such that spectral accelerations are >1.24g (spectral peak acceleration = 1.315g). See Table 1.

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Table 1

	T (sec)	S_a (g)	Participation Factor	Mode #
X-Direction (Longitudinal)	0.710	0.79	8.49	1
	0.623	0.89	9.05	2
	0.398	1.24	6.59	5
Z-Direction (Transverse)	0.589	0.94	8.28	3
	0.449	1.17	6.66	4
	0.333	1.30	4.44	6
	0.286	1.31	5.52	7
	0.235	1.28	4.54	8

Maximum displacements from the computer analysis are 3.95" in the longitudinal direction and 4.92" in the transverse direction. All analysis results are based upon elastic analysis.

Evaluation of analysis results showed that the columns are not capable of sustaining shear forces necessary to withstand the required lateral accelerations. The bridge structure was reevaluated to determine maximum probable plastic moments and corresponding shears and it was concluded that the columns will undergo a shear failure before plastic hinging will occur. Therefore, the reduction factor "Z" has been limited to 1.0. Retrofit of this structure is required if the structure must remain in service after a major earthquake. No reanalysis was performed with the modified structure, peak spectral accelerations were used for preliminary sizing of shear walls.

The capacity to resist transverse shear at the hinges has been found to be inadequate.

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6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION

6.1 No importance rating has been assigned to the structure. It is believed, however, that the rating would be low. The loss of the bridge would not have severe impact on traffic flow within the City. The type of damage that is anticipated would be lateral displacement at the hinges and column shortening. Complete collapse will not occur.

6.2 Recommendations:

6.2.1 We recommend that this structure be given a low priority for retrofit.

6.2.2 When retrofit is performed, we recommend the following:

- Provide 12 reinforced concrete shear walls as shown in Figure 2.
- Provide transverse shear restraints at the hinge locations. These restraints are shown in Figure 3.

6.2.3 An alternate solution to the concrete shear walls is encasement of the concrete columns with sufficient reinforcement to prevent shear failure. This alternate would reduce the retrofit cost by $34,000. However, we recommend the shear wall solution because:

- The column encasement will reduce but not eliminate damage.
- It is possible through the shear wall solution to virtually eliminate the possibility of damage. If money is to be expended it is prudent to strive for the most permanent solution.

6.3 Cost Estimate:

6.3.1 Shear Wall Alternate

A) Shear Walls

Item	Quantity	Unit	Unit Price	Total
Excavation	236	c.y.	20	4,720
Concrete - Wall	217	c.y.	275	59,200
Concrete - Footings	145	c.y.	200	29,000
Concrete - Pumping	362	c.y.	5	1,800
Concrete - Finishing	6,500	S.F.	0.40	2,600
Drilled Anchors	1,680	ea.	8.00	13,440
				111,260

B) Shear Plates at Hinges

Misc. Metal	10,500	lb.	2.50	26,250
Pavement Removal		L.S.		2,000
Pavement Patching		L.S.		2,500
Drillholes	140	ea.	20.00	2.800
1-1/4" Bolts-installed	140	ea.	10.00	1,400
				34,950

Subtotal		146,210
Mobilization		10,000
Contractors OH&P (20%)		31,200
Escalation (8%)		15,000
Subtotal		202,400
Engineering - 15%	=	30,400
TOTAL	=	$232,800

6.3.2 Alternate to Shear Wall Construction

A) Encase Columns:

Item	Quantity	Unit	Unit Price	Total
Excavation	40	c.y.	20	800
Sandblast	2,900	s.f.	3.00	8,700
4" Gunite	3,100	s.f.	12.00	37,200
Finishing	3,100	s.f.	1.00	3,100
Reinforcing	21,000	lb.	0.75	15,750
Welded Spices	1,080	ea.	11.00	11,900
				77,450

B) Shear Plates at Hinges:	34,950
Subtotal	112,400
Mobilization	10,000
Contractors OH&P (20%)	24,500
Escalation (8%)	11,750
Subtotal	158,650
Engineering (20%) =	31,000
TOTAL =	$ 189,650

Related Drainage Control Work:

Check and clean weep holes	$2,000
Engineering supervision and inspection	1,000
Total	$3,000

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7. SUMMARY AND CONCLUSIONS

The Bryant Street Viaduct between Second Street and Beale Street is an eight span structure approximately 442 feet long. An additional 276 feet on the north end is constructed on fill placed between retaining walls.

The superstructure is supported by two parabolic longitudinal girders. Transverse beams are approximately eleven feet apart.

The anticipated peak rock acceleration is 0.5g and the maximum spectral acceleration is 1.3g.

Damage to the structure would not be catastrophic. Traffic is light and alternate routes are available. The remedial work prepared may be given low priority.

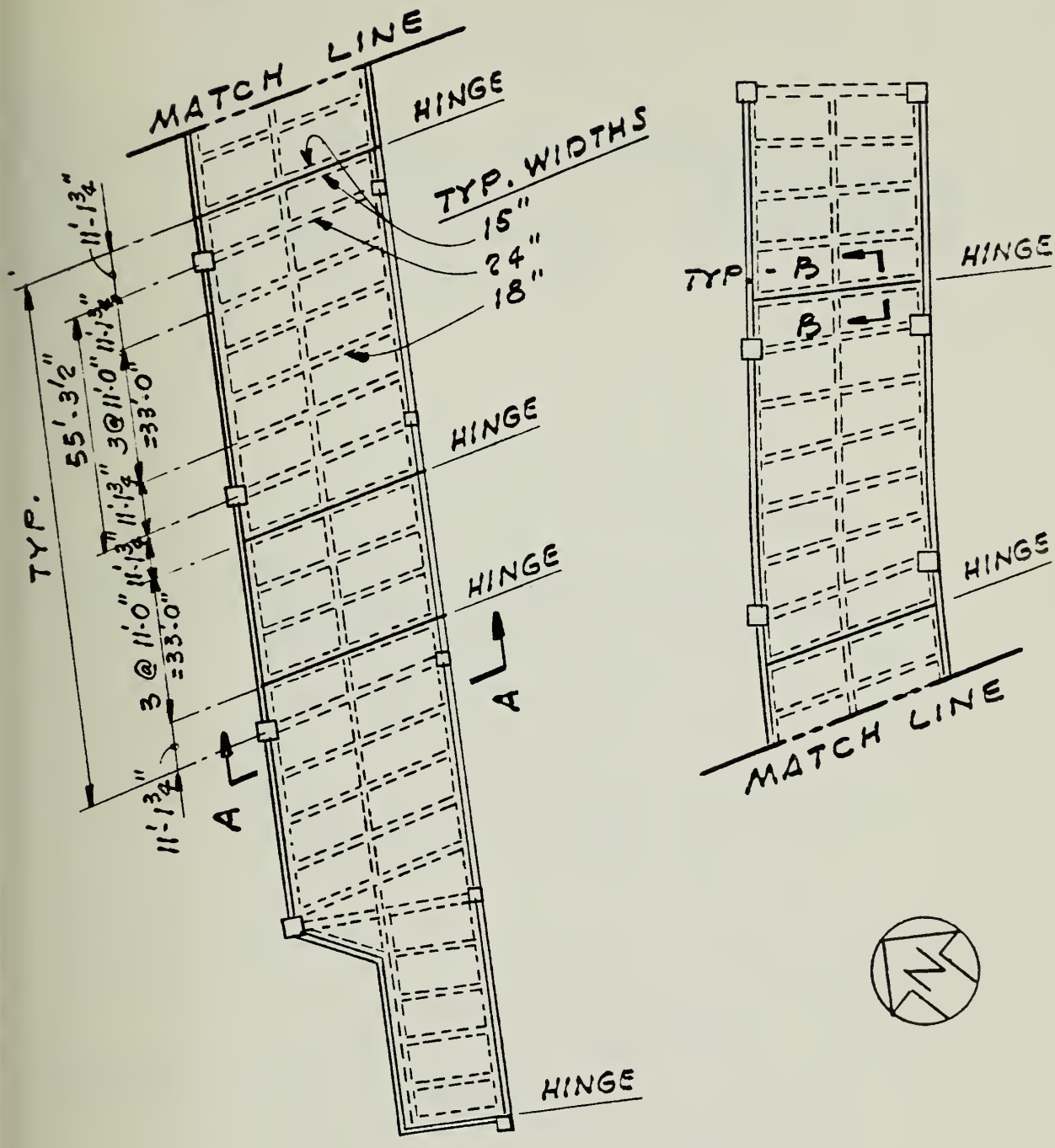
Anticipated damage would be shear failure of columns and transverse displacement at expansion joints. The column shear failure is not expected to cause collapse of the structure because of the low height to thickness ratio of the four foot square columns.

The recommended remedial work consists of:

- Constructing transverse and longitudinal shear walls between columns.
- Installing lateral restraints in the deck at the expansion joints.

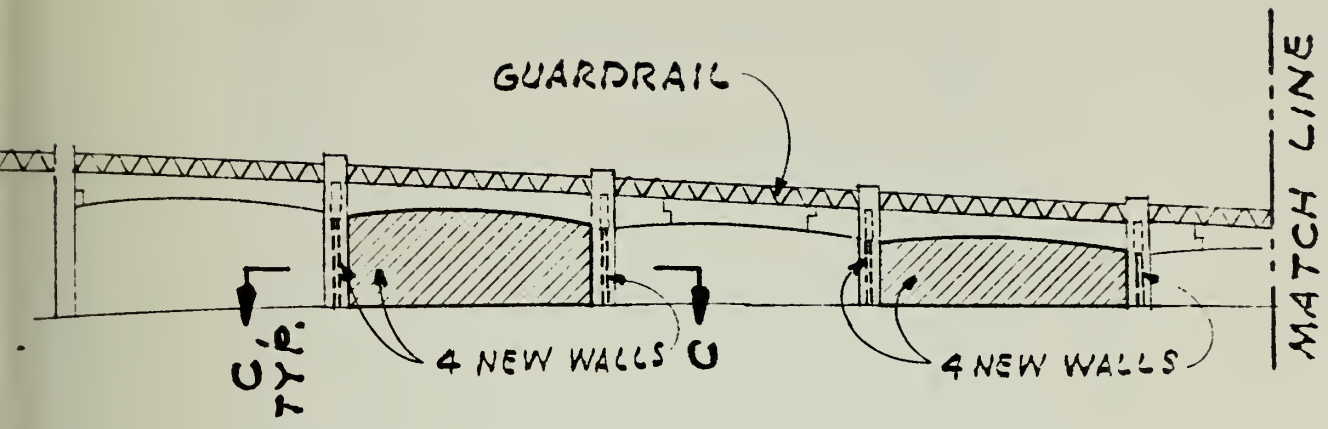
The above measures would virtually eliminate potential damage to the structures.

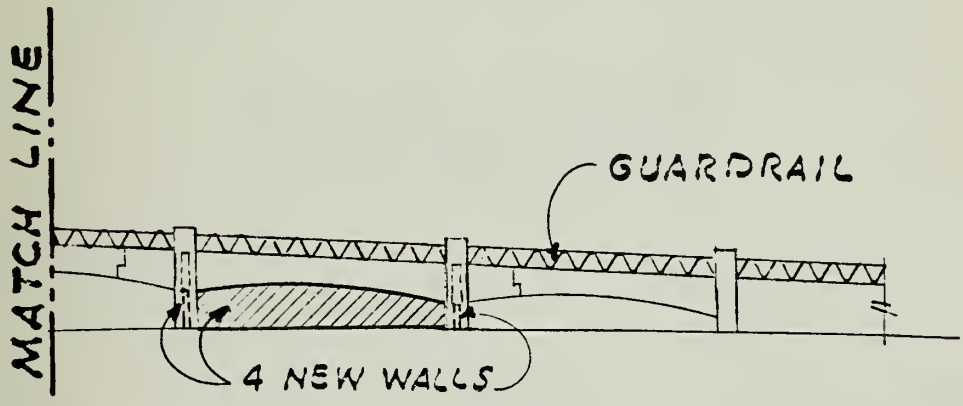
Column encasement is suggested as a less costly alternative to shear walls. However, the shear walls provide a more permanent solution and the additional cost is justified.



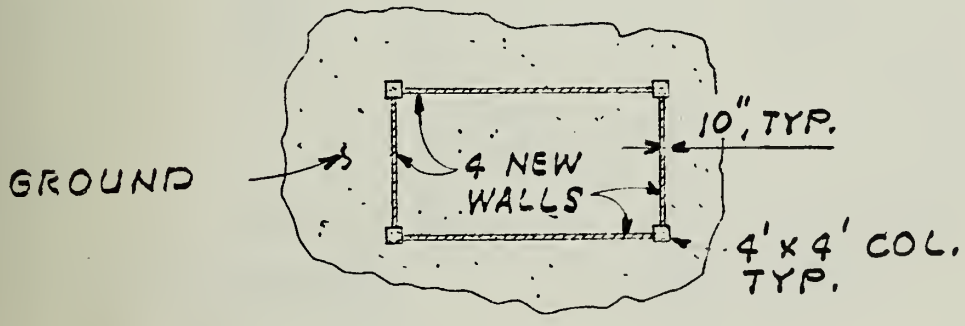
PLAN

PLAN
BRYANT ST. VIADUCT BETWEEN 2ND & BEALE
S.F. BRIDGE GROUP II A-8 FIGURE 1



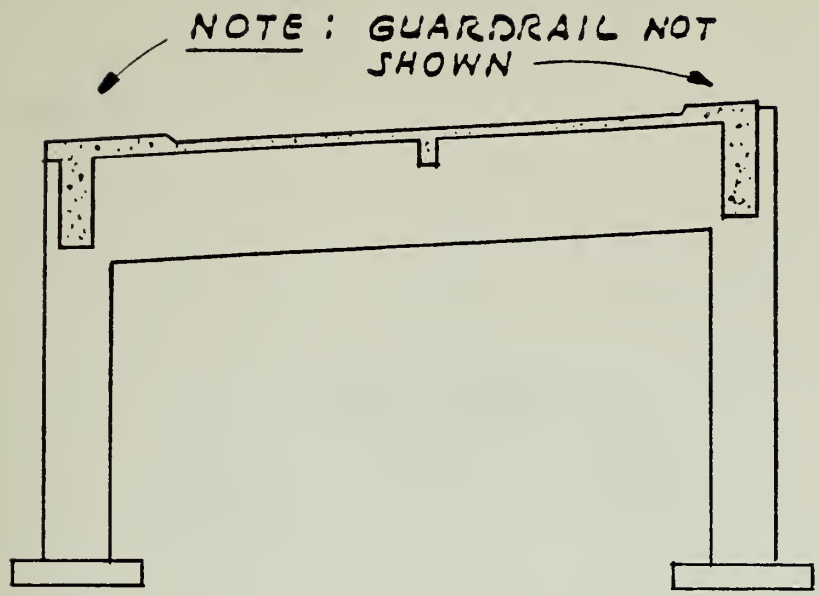


ELEVATION

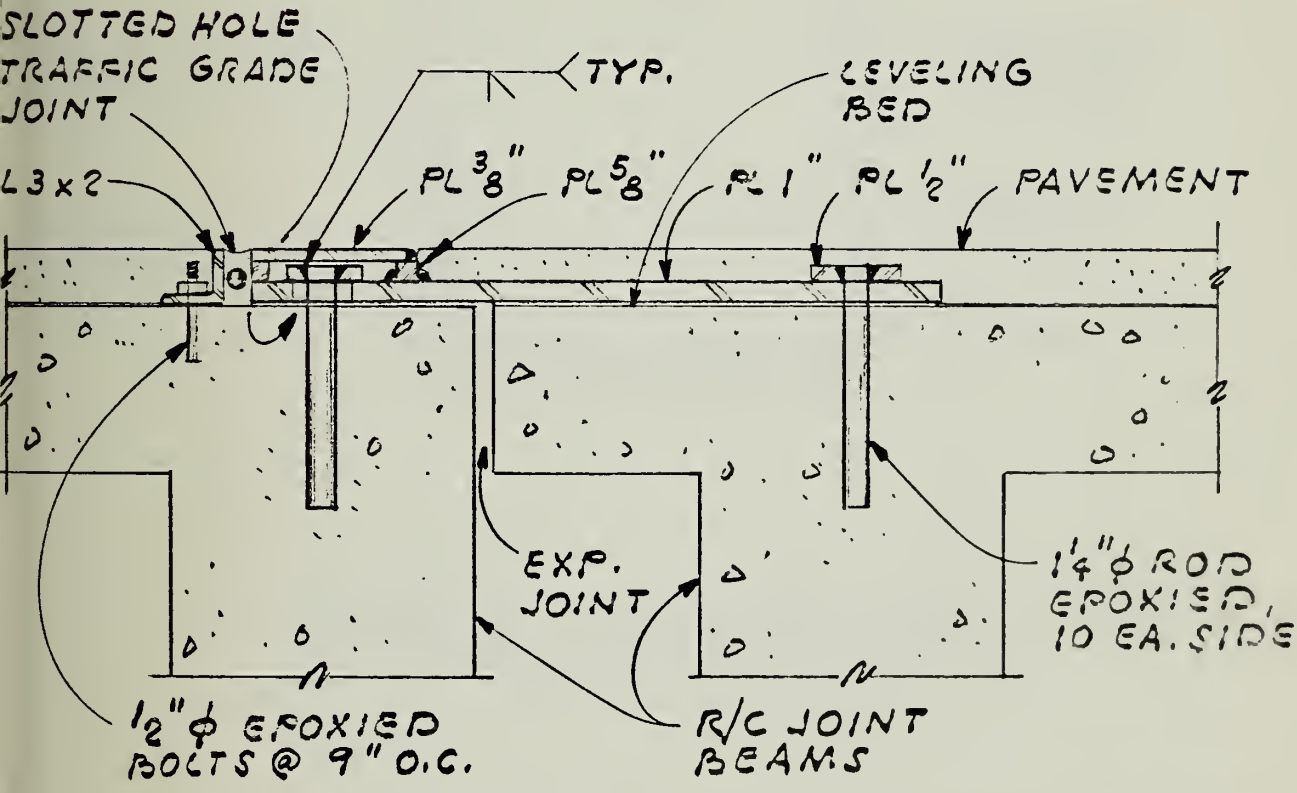


SECTION C-C

ELEVATION AND SECTION
BRYANT ST. VIADUCT BETWEEN 2ND & BEALE
S.F. BRIDGE GROUP IIA-8 FIGURE 2

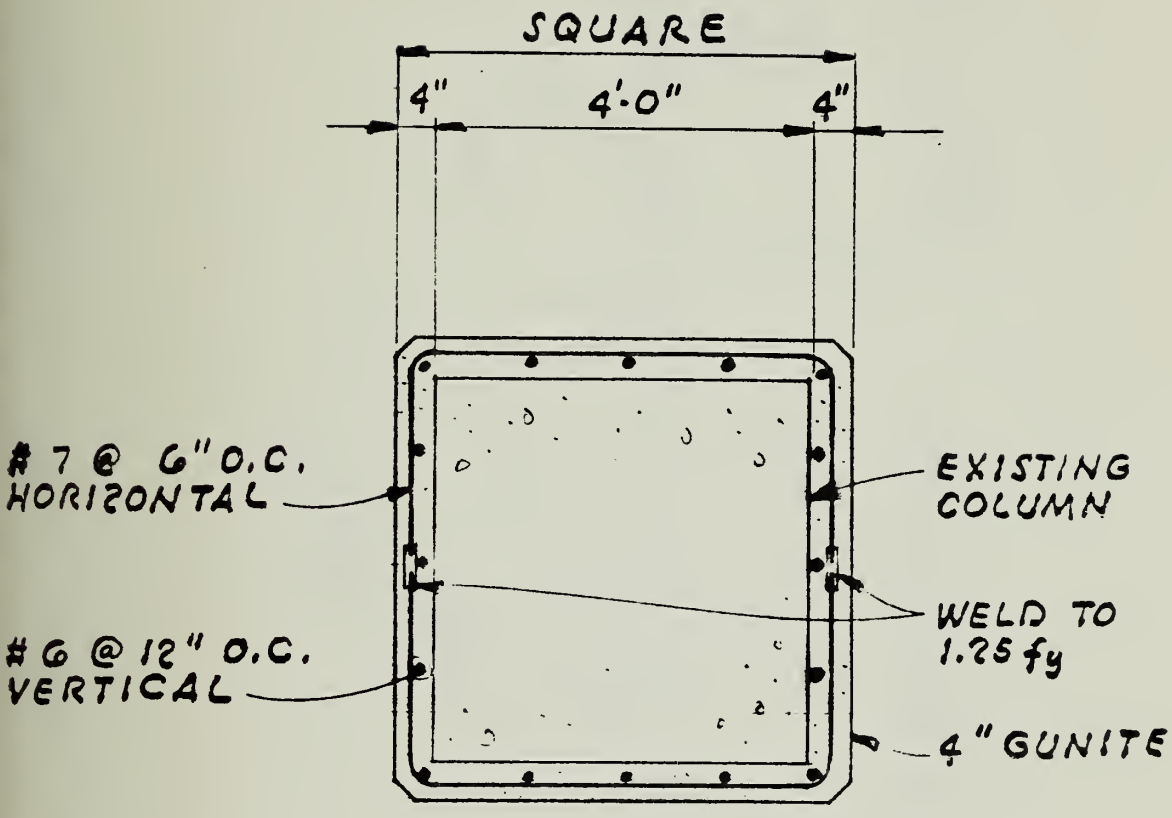


TYPICAL FRAME
SECTION A-A



SECTION B-B THROUGH MODIFIED HINGE
TYP. 3 LOCATIONS

SECTIONS
BRYANT ST. VIADUCT BETWEEN 2ND & BEALE
S.F. BRIDGE GROUP II A-8 FIGURE 3



COLUMN ENCASEMENT
TYPICAL ON 12 COLS.

LUMN ENCASEMENT ALTERNATE TO SHEAR WALLS
YANT ST. VIADUCT BETWEEN 2ND & BEALE
F. BRIDGE GROUP II A-8 FIGURE 4

OMPUTER MODEL
YANT ST. VIADUCT BETWEEN 2ND & BEALE
F. BRIDGE GROUP II A-8

FIGURE 5

MISSION STREET VIADUCT OVER ALEMANY BLVD.

IIA-9

MISSION STREET VIADUCT AT ALEMANY BOULEVARD

CONTENTS

		page
1.	INTRODUCTION	IIA-9.1
2.	SITE INVESTIGATION	IIA-9.2
3.	BRIDGE DESCRIPTION	IIA-9.4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING BRIDGE	IIA-9.6
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE AND MODIFIED BRIDGE	IIA-9.8
6.	RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION	IIA-9.10
7.	SUMMARY AND CONCLUSIONS	IIA-9.11
	REFERENCES	APPENDIX C

FIGURES:

1.	PLAN AND LONGITUDINAL SECTION	IIA-9.12
2.	PROPOSED STEEL SHEAR LUGS	IIA-9.13

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MISSION STREET VIADUCT AT ALEMANY BOULEVARD

1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

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2. SITE INVESTIGATION

Results of the site investigation for all bridges is given in Reference 3. A general summary of the reference as it pertains to the Mission Street Viaduct at Alemany Boulevard is given in this section.

This viaduct is a cast-in-place concrete vehicular bridge along Mission Street crossing over Alemany Boulevard.

Available geological information indicates that the site is underlain by Colma Formation (Qc). The underlying bedrock is Franciscan Formation of the Central Highlands belt. The depth to the bedrock is estimated to be approximately 50 ft. An exploratory boring, DH-15, was drilled on Trumbell Avenue, near the southern pier, to explore the subsurface conditions. The boring was 21 ft deep, and the materials encountered consisted of approximately 9 ft of artificial fill, dense silty sand (SM), underlain by very dense, medium-grained, poorly graded Colma Formation (SP) extending to the depth explored. Standard blow counts of artificial fill were 35 and 49 per foot of penetration. Standard blow counts of Colma Formation were 91 and 115 per foot of penetration. Ground water was not encountered in the boring.

The viaduct is located at approximately 5.5 mi northeast of the San Andreas fault and 13.5 mi southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g. The maximum ground surface velocity is estimated to be approximately 24 in. per second. Response Spectrum I, shown on Figure 2 of Reference 3, and Response Spectrum II, shown in Appendix B, are recommended for the seismic safety evaluation.

The high density of the soil underlying the viaduct suggests that liquefaction is not likely. Plate 10 of Reference 3 shows that the viaduct is located in the area of potential landslide. Due to the high density of the underlying soil, however, the risk associated with landslide is minimal. The site is located at a great distance from the areas of potential lateral spreading and tsunami inundation, as shown on Plates 3 and 11 of Reference 3.

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Some settlement, however, was noticed at the southwestern abutment. The settlement is believed to be caused by compaction of sandy fill, and it has caused some cracks of the bridge at various locations. We recommend that the viaduct structure be thoroughly inspected and all cracks repaired. Additional settlement and cracking may be anticipated during a major earthquake.

3. DESCRIPTION OF BRIDGE

3.1 Location:

The Mission Street Viaduct carries Mission Street over Alemany Boulevard. It is located immediately south of Interstate 280 and 1.4 mi west of Highway 101.

3.2 Physical Description:

The viaduct is a two-span continuous girder bridge supporting a 58-ft-6-in. wide roadway and two 12-ft-0-in. wide sidewalks. Each span is 89 ft 7 in. in length.

The superstructure consists of six reinforced concrete girders supporting a cast-in-place deck The center pier and the abutments form an angle of 32° with the bridge centerline. The overall length of the center pier is 147 ft 6 in. The clear distance between abutments is 90 ft.

The bridge is actually three separate structures. The center portion, 20 ft wide, was designed as a railroad structure. The outer roadway structures are physically separated by an expansion-material filled joint.

The bridge was constructed in 1950 on the site of an existing bridge, and some of the existing piles were reused. Existing wing walls and the walks between the wing walls and new abutment walls were also incorporated into the new structure.

3.3 Traffic Flow:

Mission Street is a main artery between the Mission District on the north and Daly City-Colma to the south. However, most through traffic now follows Interstate 280, leaving local traffic on Mission. Alemany Boulevard serves as a frontage road to Interstate 280 east of Mission, and then turns and parallels Mission on the west. As a result, it carries relatively light traffic.

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3.4 Description of Bridge Condition:

3.4.1 The substructure is in good condition, except that some cracking has occurred at the southwestern abutment which is believed to be caused by settlement of the sandy fill.

3.4.2 The beams and slabs are in good condition. The solid concrete balustrade on the east side, however, has developed cracks near the bridge centerline and over the north span. Since the balustrade is not a structural element of the bridge, the cracks may be considered to be cosmetic.

3.5 Description of Specialty Items:

3.5.1 The superstructure girders are supported on the center bent with a direct concrete-to-concrete bearing. Interior girder bearings are 2 ft 6 in. wide by 3 ft 8 in. long and are anchored with eight 1-1/4-in. square dowels. Exterior girder bearing areas are 3 ft 0 in. wide by 2 ft 8 in. long with eight 1-1/4-in. square dowels.

3.5.2 The abutment ends of the girders are supported on steel rocker bearings which are built into recesses in the abutment. The bearings are completely concealed and cannot be inspected without breaking out part of the abutment.

4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE

4.1 The criteria for analysis of the existing bridge, as well as for the modified bridges, are given in Appendix A. A general summary of the appendix as it pertains to the Mission Street Viaduct at Alemany Boulevard is given below:

4.1.1 The soil investigation shows that the bridge is situated on alluvium with 50-ft depth to bedrock. The Caltrans Response Spectrum II, shown in Appendix B, was used for seismic safety evaluation. The peak rock acceleration (PRA) is 0.5g, the peak ground acceleration (PGA) is 0.7g, and the damping is taken as 5% of critical.

4.1.2 The structure has been checked for the following two loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

Load Case 2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

where

U = Total load to be resisted

$D.L.$ = Dead load

$E.Q.X$ = Earthquake load in X-direction (longitudinal)

$E.Q.Z$ = Earthquake load in Z-direction (transverse)

Z = Adjustment factor for ductility and risk

4.1.3 The adjustment factor Z for ductility and risk for concrete members and connections are as follows:

Component	Z
Single column pier	2*
Wall-type pier	2*
Abutment connection	0.8
Expansion joint	0.8
Column pier connections (to superstructure or foundation)	1.0

*The Z factor is to be used for modifying the bending moment of the column. For shear or axial force, a Z-factor of 1.0 shall be used.

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4.1.4 Ultimate stresses for the existing construction are taken as follows:

Concrete f'_c = 2,500 psi

Reinforcing Steel f_y = 40,000 psi*

*The steel grade is not specified in the plans. However, based on the 1950 construction date, it is reasonable to assume that A15 Grade 40 steel was used.

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5. SEISMIC ANALYSIS OF EXISTING AND MODIFIED BRIDGE

5.1 Analysis Method:

The Mission Street Viaduct was analyzed for seismic forces using the computer program SEISAB, Version 1.3,written by the Engineering Computer Corporation (Reference 5).

5.2 Assumptions:

5.2.1 Potential collapse mechanisms of this bridge are limited to torsional movement about a vertical axis in the clockwise direction. Longitudinal movements will result in a large transverse force as the girder reacts against the sharply skewed abutment back wall. One 18-in. high key is provided above the abutment adjacent to each interior roadway girder to resist this transverse motion.

5.2.2 The abutments are massive and receive support from the wing walls and a system of transverse tie walls. They are assumed to move with the ground and were not analyzed.

5.2.3 The superstructure was assumed to be able to move longitudinally on the rocker bearings at the abutment, but was considered to be restrained against lateral movement at the abutment. One run was made assuming fixity between the center bent and the bridge girders, while a second run assumed a pinned condition between the center bent and the bridge girders.

5.2.4 The recommendations for retrofit are intended to prevent lateral displacement of the deck at the abutments. Therefore, the assumptions made in the initial analysis became the true conditions, and an "as modified" analysis was not necessary.

5.3 Result of Analysis:

5.3.1 The predominate periods of vibration in the two directions, along with their corresponding spectral accelerations (S_a) and participation factors, are given below.

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	T (sec)	S_a (g)	Participation Factor	Mode
Longitudinal	0.127	1.40	6.184	3
(x-direction)	0.066	1.13	4.34	6
Transverse	0.127	1.40	5.217	3
(z-direction)	0.066	1.13	5.257	6

The second mode was vertical deflection. The first, fourth, and fifth modes had zero participation factors.

5.3.2 The computer model indicates the following elastic displacements:

Longitudinal 0.15 in.	0.15 in.
Transverse (at center bent)	0.18 in.

5.3.3 The elastic analysis assigns about 2,150 kips force normal to the center pier. This requires a Z-factor of 7, which is normally not permissible with nonductile detailing. However, the small longitudinal displacement is indicative that very little damage could occur in the connections. The bridge will perform satisfactorily if displacements at the abutment supports are limited.

The dowels connecting the center pier to the foundation limit the transverse moment capacity of the base of the pier to 4,650 ft-kips, or a maximum transverse force (normal to the pier) of 310 kips.

A static analysis was performed using the peak of the response spectrum (1.64), modified by a Z-factor of 2. The Z-factor is appropriate because the restrainers at the abutments will permit some displacement. The total seismic force, less the 310 kips taken by the center pier, is assigned to the abutment restrainers.

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6. RECOMMENDATIONS AND COST ESTIMATE FOR STRUCTURAL MODIFICATION

6.1 Recommendations:

6.1.1 The purpose of the modifications of the abutments is to prevent transverse displacement of the bridge deck at the abutment rocker bearings.

Displacement to the left (facing the abutment) is prevented by the highly skewed back wall of the abutment.

Restrainers must be added to prevent displacement to the right. Steel buttress plates, bolted to the top of the abutment seat, are recommended. The buttress plates are detailed in Figure __.

6.2 Cost Estimate:

Item	Description	Quantity	Unit	Unit Price	Total
1.	Mobilization		L.S.		$5,000
2.	Traffic Control		L.S.		6,000
3.	Abutment Shear Lugs:				
	Drill 1'4" Holes	96	ea.	50.00	4,800
	Bolts	96	ea.	40.00	3,840
	Metal Fabrication	6,400	lb.	5.00	32,000
	Grouting	8	ea.	500.00	4,000
	Subtotal				$55,640
	Contractor's OH&P (20%)				11,130
					$66,770
	Contingency (20%)				13,330
	Engineering				16,000
	Escalation (8%)				6,400
	TOTAL				$102,500

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6.3 Related Work (Low Priority):

Alleviate potential settlement at southwest abutment by pressure grouting:

Grouting	$15,000
Engineering supervision and inspection	3,000
Total	$18,000

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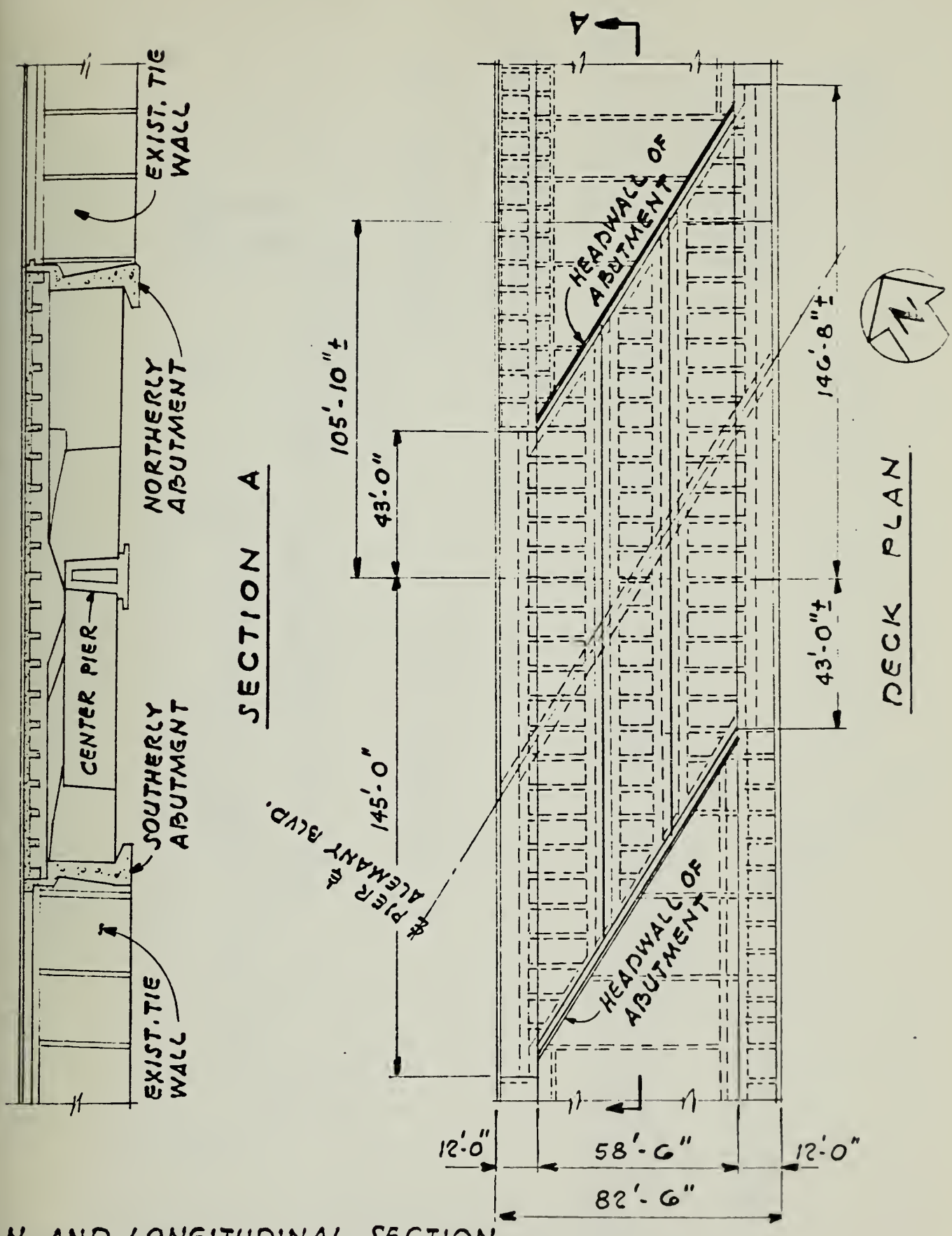
7. SUMMARY AND CONCLUSIONS

The Mission Street Viaduct over Alemany Boulevard is inherently a rigid structure. However, the 58° skew of the abutments and center bend create force components which force the structure into clockwise rotation, and there is inadequate restraint to prevent this rotation.

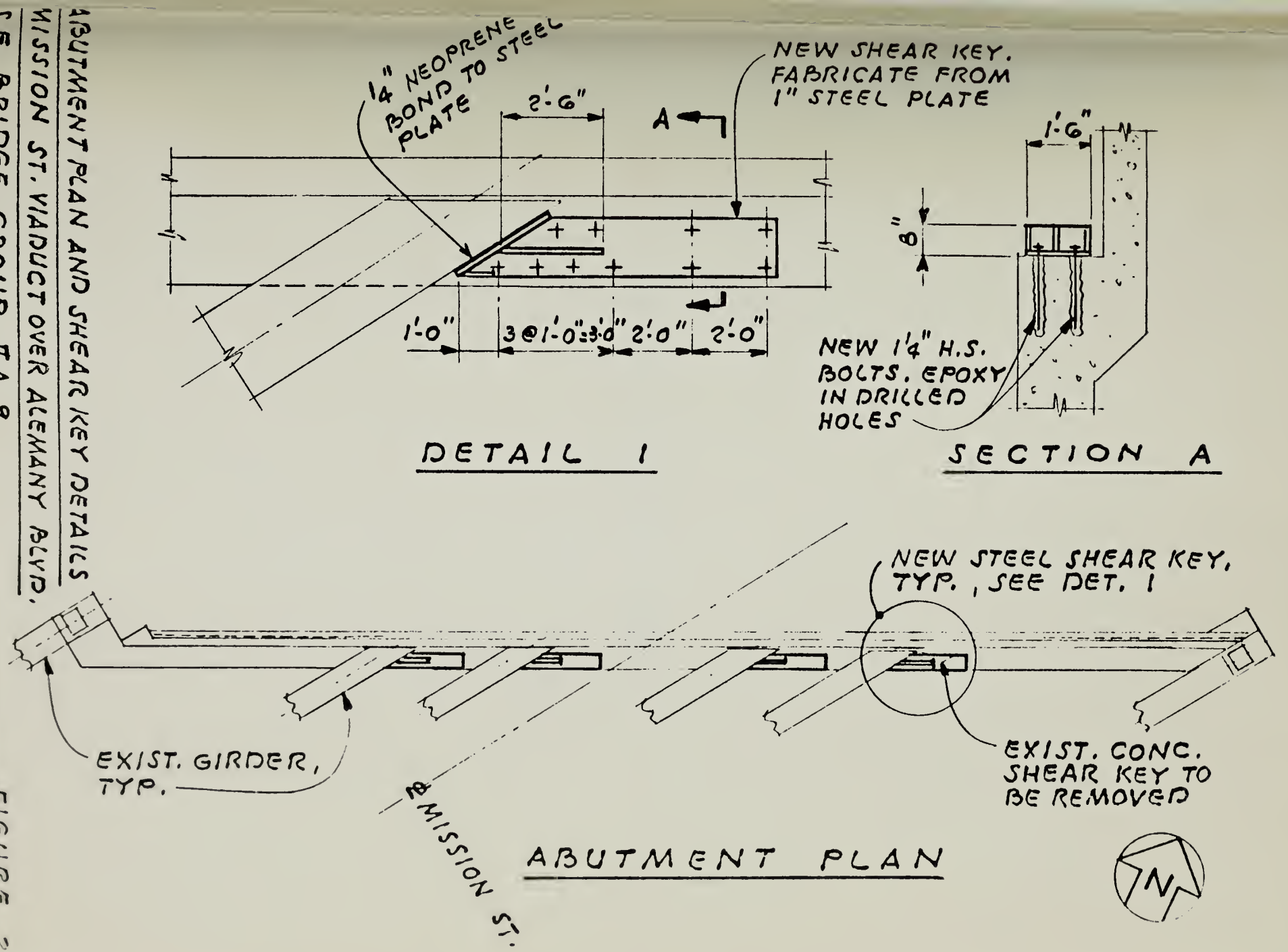
Remedial measures proposed are to place neoprene-faced steel buttress plates on the abutment seat adjacent to the bridge girders.



...AN AND LONGITUDINAL SECTION

...ISSION ST. VIADUCT OVER ALEMANY BLVD.

...F. BRIDGE GROUP IIA-9 FIGURE 1



ABUTMENT PLAN AND SHEAR KEY DETAILS
MISSION ST. VIADUCT OVER ALEMANY BLVD.
S.F. BRIDGE GROUP II A-9

FIGURE 2

THIRD STREET BRIDGE OVER CHANNEL

V-1

THIRD STREET BRIDGE OVER CHANNEL STREET WATERWAY

CONTENTS

		page
1.	INTRODUCTION	V1-1
2.	SITE INVESTIGATION	V1-2
3.	BRIDGE DESCRIPTION	V1-4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE	V1-8
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE	
	5.1 Bridge in Closed Condition	V1-10
	5.2 Bridge in Open Condition	V1-15
6.	RECOMMENDATIONS AND COST ESTIMATES FOR REMEDIAL WORK	V1-17
7.	SUMMARY AND CONCLUSIONS	V1-18
	REFERENCES	APPENDIX C

TABLES

1	Section Properties - Beam Elements	V1-6
2	Results of Seismic Evaluation - Truss Elements	V1-13
3	Results of Seismic Evaluation - Beam Elements	V1-14

FIGURES

1	Elevation and Plan	V1-5
2	Mathematical Models	V1-11

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1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

2. SITE INVESTIGATION

Results of the site investigation for all bridges are given in Reference 3. A general summary of this reference as it pertains to this bridge under consideration is given in this section.

The bridge is located along the 3rd Street crossing over Channel Street Waterway. Available geological information indicates that the site is underlain by artificial fill deposits (Qaf). The underlying bedrock is Franciscan formation of the Nob Hill belt. The depth to the bedrock is estimated to be approximately 120 ft. An exploratory boring, DH-9, was drilled at the southern approach of the bridge to explore the subsurface conditions. The boring was 30 ft deep and the materials encountered consisted of approximately 10 ft of artificial fill, dense silty sand (SM) underlain by soft younger bay mud (CL) extended to the depth explored. Standard blow counts of the upper sand (SM) were 41 and 46 per ft of penetration. Soft younger bay mud (CL) had a blow count of 1 per ft of penetration and a torvane undrained shear strength of 300 psf. Ground water was measured at a depth of 6 ft. The thickness of the bay mud is estimated to be 50 ft, based on other information.

The bridge is located approximately 9 mi northeast of the San Andreas fault and 10 mi southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g and 0.3g, respectively. The maximum ground surface velocity is estimated to be approximately 15 in. per second. Response Spectrum III, shown on Figure 2 of Reference 3, and Response Spectrum IV, shown in Appendix B, reduced by 20%, are recommended for the seismic safety evaluation.

Plate 9 of Reference 3 shows that the bridge is located in an area of potential liquefaction. The exploratory boring drilled at the southern approach of the bridge indicates that the artificial fill consists of dense silty sand. Borings explored for other bridges at the vicinity of the site, however, disclosed loose saturated sandy fill. It is likely such materials are also present at this site. Accordingly, liquefaction may still be a problem at the site. Although the bridge is pile supported, liquefaction of the fill material would result in some structural damage of the bridge. The

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traffic will be disrupted. Soil liquefaction may be alleviated by grouting of the loose sandy material. Cement and/or chemical grouting of the sandy material would increase its cohesive strength and thus reduce its liquefaction potential. Alternatively, well drained stone columns or wick drains may be placed at about 5 ft center to center with a gravel drainage blanket beneath the pavement. the stone column and wick drain solutions may alleviate the soil liquefaction but result in several inches of settlement after a major earthquake. Additional field exploration is required to define the extent of the loose material and its thickness.

The bridge lies on a relatively flat ground, and the risk of landslide is minimal. Localized slope failure towards the channel, however, may occur during a major earthquake. The problem of slope instability may be alleviated by placement of rip-rap and filter materials.

The bridge is located outside the pre-1860 shoreline, as shown on Plate 3 (Reference 3), where soil lateral spreading may occur during a major earthquake. The boring information, however, indicates that the fill over bay mud is dense to very dense, and the fill has been placed for many years. Large-scale lateral spreading of the bridge site is considered unlikely. The risk of lateral spreading will be significantly reduced if the liquefaction potential is decreased by the above recommended solutions.

The bridge is located in the area of potential tsunami inundation, as shown on Plate 11 (Reference 3).

3. BRIDGE DESCRIPTION

This bridge is a single-leaf bascule steel structure with reinforced concrete slab approach spans. the structure was designed in 1932. The bridge is supported by three piers and is oriented in the northerly direction. The piers are supported by vertical and battered wood piles. The bridge has a width of 80 ft and the maximum span of 142 ft between piers No. 2 and No. 3. Elevation and plan of the bridge are illustrated in Figure 1.

The south approach is made up of reinforced concrete slab and 4 wall-type girders spanned at approximately 24 ft, supported on 18 in. by 18 in. vertical concrtete piles. The north approach between piers No. 1 and No. 2 consists of reinforced concrete slab on steel girders. The movable steel bascule truss can be rotated around the main trunnion with the help of two concrete counterweights (CT.WT.) of 1,370 kips and 1,060 kips in the west and east trusses, respectively. When the bridge is in a closed position, its south end of the bascule truss rests on the concrete pier (pier No. 3). It has a restrainer to prevent movement in the transverse direction, but allows free movement in the longitudinal direction.

The steel portion of the bridge includes two bascule trusses spanned at 53 ft-8 in. center to center, two main trunnion posts, two towers, and two CT.WT. trusses. There are two link members connecting the bascule and CT.WT. trusses. The floor system consists of open steel deck, stringers, built-up steel girders, and diagonal bracing members. Unlike the 3rd Street Bridge, the east and west bascule trusses are tied and braced with steel members. Table 1 (page VI-6) and Table 2 (page VI-13) show member sizes for representative steel members.

Some structural alterations and repairs have been made to the bridge since its completion in 1933. In 1953, a concrete mass was added to the south end girder to counterbalance the removal of the east-west trolley arch on the south end of the moving leaf. In addition, the steel grating was installed to replace the wooden floor system. In 1971, the floor girders in the north approach and some bracing members of the floor system were rehabilitated. All metal surfaces were painted and some minor repairs were made in 1974.

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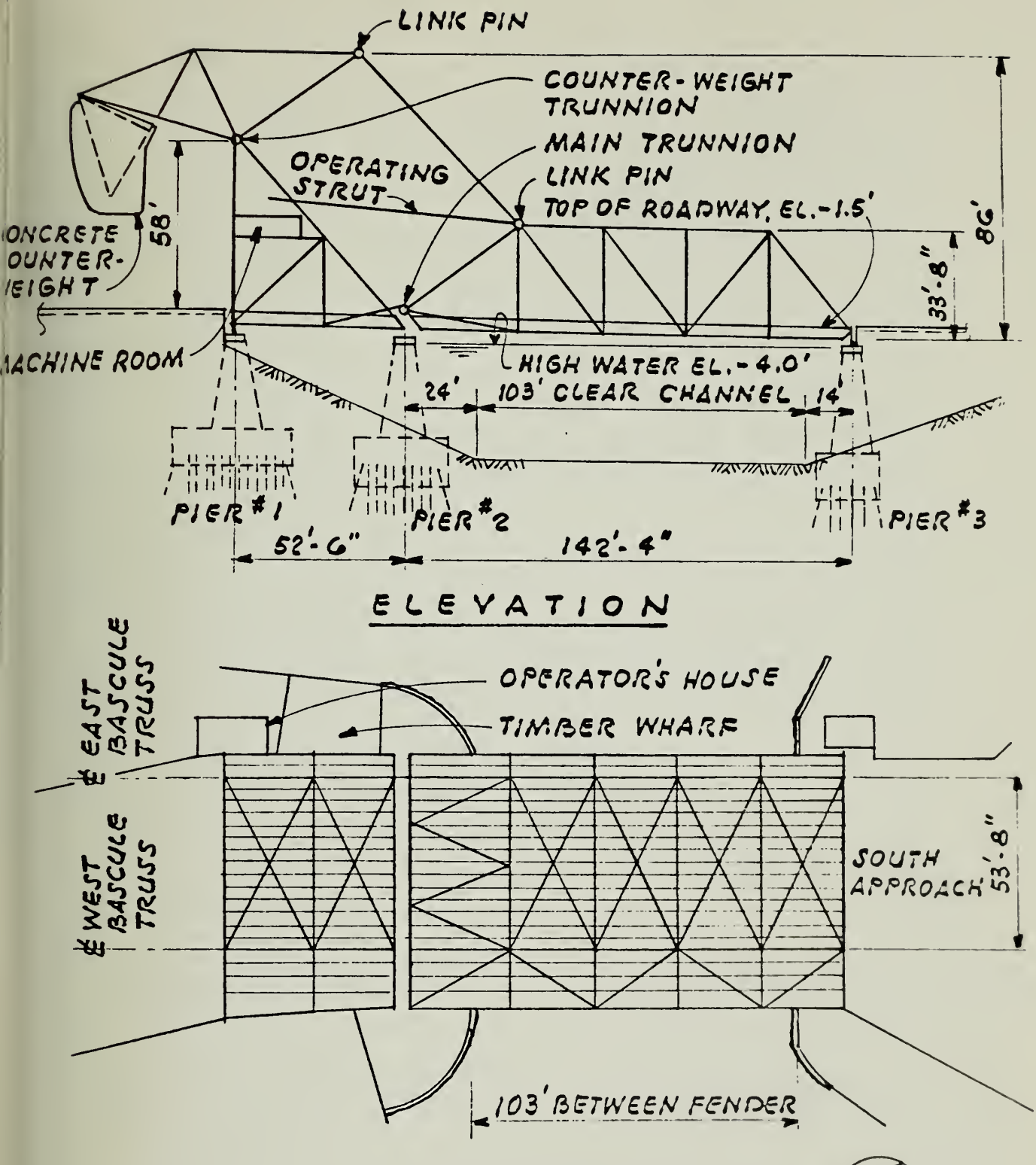
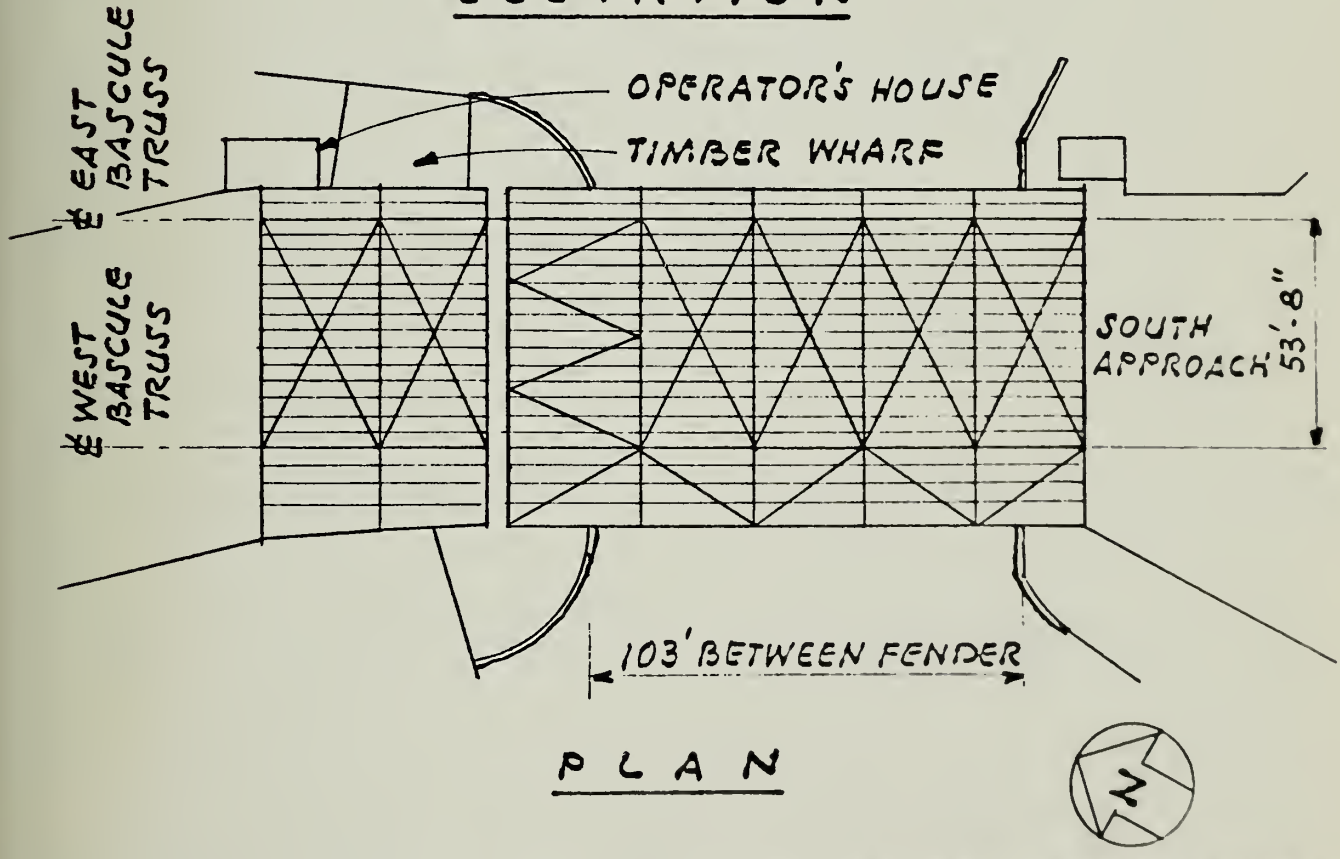


ELEVATION



PLAN

ELEVATION AND PLAN OF 3RD STREET BRIDGE OVER CHANNEL

S.F. BRIDGE GROUP V-1 FIGURE 1

over channel (Beam Elements)

Member* I.D.	Nodal* Point	A (in^2)	S_{2-2} (in^3)	S_{3-3} (in^3)	F_a (Ksi)	F_b (Ksi)	Section**
1	1 - 3	31.4	-	472	18.0	18.0	4-L4 x 4 x 3/8, 2-PL 20 x 1/2
7	3 - 7	31.4	-	472	18.0	↑	"
11	9 -13	37.5	-	562	16.0		4-L4 x 4 x 1/2, 2-PL 20 x 9/16
13	11-15	31.4	-	472	18.0		4-L4 x 4 x 3/8, 2-PL 20 x 1/2
14	12-16	31.4	-	472	18.0		"
15	13-17	60.0	-	900	16.0		4-L4 x 4 x 1/2, 4-PL 20 x 9/16
18	18-20	23.6	-	684	13.8		2-C15 x 40
21	15-27	31.4	-	477	13.9		4-L4 x 4 x 3/8, 2-PL 20 x 1/2
22	16-28	41.7	-	626	14.8		4-L4 x 4 x 9/16, 2-PL 20 x 5/8
23	23-27	79.5	913	-	18.0		6-L6 x 6 x 1/2, 1-PL 23 x 1/2
25	23-33	97.5	-	1244	15.7		4-L6 x 6 x 9/16, 4-PL 28 x 1 1/4
29	37-39	300.0	-	3611	16.6		8-L 6 x 6 x 5/8, 6-PL 36 x 1
37	15-29	84.8	1550	-	18.0		4-L4 x 4 x 1/2, 2-PL 20 x 7/16
39	29-31	121.0	812	-	14.8		4-L4 x 4 x 11/16, 2-PL 30 x 1 1/2
42	30-44	100.7	678	-	18.0		4-L6 x 6 x 9/16, 2-PL 30 x 1 1/4
43	31-43	88.0	507	-	15.8		4-L6 x 6 x 1/2, 2-PL 26 x 1 1/4
44	32-44	88.0	507	-	15.8		4-L6 x 6 x 13/16, 2-PL 26 x 5/8
45	43-45	71.3	580	-	18.0		4-L6 x 6 x 1/2, 2-PL3 30 x 1 1/8
48	44-46	60.9	-	1332	18.0	↓	4-L6 x 6 x 5/8, 2-PL 26 x 5/8
51	31-45	132.4	971	-	16.7	18.0	4-L6 x 6 x 13/16, 2-PL 32 x 1 1/2

* See Figure 2 for member location

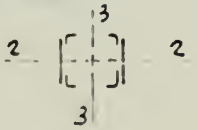
** Typical section 

TABLE 1

When the bridge is subjected to an earthquake in the longitudinal direction, the lateral force induced from the CT.WT. mass would be carried through the link members and the CT.WT. tower to the bridge piers. In the transverse direction, the horizontal ties and bracing members between the east and west trusses would help to tie both together as a coupled system. The lateral force-resisting system of the floor deck consists of diagonal bracing to resist lateral forces in both the longitudinal and transverse directions.

Field inspection of the bridge reveals that the steel portion of the bridge on the roadway and above is generally in good condition. No apparent structural deficiencies regarding riveted joints and CT.WT. connections were observed. Note that the structural conditions under the bridge floor, as well as the substructures, were not subjected to field investigation in this study. However, based on the field inspection report conducted by Caltrans in 1982, the following structural deficiencies were found:

1. The reinforced concrete south approach is in very poor condition. Some of the concrete girders have large spalls with exposed reinforcement.

2. The structural steel in the lower portions of the girder near the bearing areas at the southeast corner of the movable deck are heavily corroded.

The condition of the timber piles has not been investigated. However, experience has shown that timber piles below the mud line are not subjected to the normal causes of deterioration. Although the geotechnical report suggested that they be inspected, we do not believe this is necessary.

The average daily traffic flow over the bridge was estimated to be approximately 19,000, based on the available information published by Caltrans in 1982. As for the traffic flow under the bridge, an interview with the bridge operator indicates that the bridge is lifted about 4 to 5 times every day and 4 to 5 minutes during each operation.

4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE

The criteria for analysis of the existing structure, as well as for the modified structure, are given in Appendix A. A general summary of the appendix as it pertains to the bridge discussed here is given below:

1. Because the bridge is located in an area with soft bay mud, a Caltrans response spectrum IV, shown in Appendix B, reduced by 20%, will be used for seismic safety evaluation. The peak ground acceleration (PGA) of the response spectrum is 0.4g, with a damping of 5% critical.

2. The existing and modified structures will be checked for the following four loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

3: $U = D.L. + (E.Q.X + E.Q.Y)/Z$

4: $U = D.L. + (E.Q.Z + E.Q.Y)/Z$

where

U = Total load to be resisted

$D.L.$ = Dead load

$E.Q.X$ = Earthquake load in X-direction (longitudinal)

$E.Q.Y$ = Earthquake load in Y-direction (vertical)

$E.Q.Z$ = Earthquake load in Z-direction (transverse)

Z = Adjustment factor for ductility and risk

3. The adjustment factor Z for ductility and risk for structural steel members and connections are as follows:

Member	Z
Beam	3
Top & Bottom Chords of Truss	3
Column & Bracing	2
Connection	1.5

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4. Allowable stresses for both new and old construction will be taken as follows:

Members	1.7 F_s
Connections	1.3 F_s

The term F_s for the existing steel members is the allowable working stress based on F_y = 30 ksi. Similarly, F_s for new steel members is the allowable working stress based on F_y = 36 ksi. Both levels of working stresses are given in the AISC Manual of Steel Construction, eighth edition.

5. Except as noted in the drawings, the connections will be assumed to be 7/8-in.-diameter rivets with an allowable shear capacity of 10 kips for each rivet.

5. SEISMIC ANALYSIS OF THE EXISTING BRIDGE

5.1 Bridge in Closed Condition:

Linear and three-dimensional finite element response spectrum analyses were undertaken using the SAP IV computer program (Reference 4). Figure 2 represents the superstructure of the bridge. The model consists of 82 beam elements and 72 truss elements with nodal masses applied at appropriate nodes. The foundation springs simulating the pile and pier foundations were modeled at the supporting piers as shown in the figure. The pile foundation springs were calculated according to the procedure suggested in Reference 6. The horizontal foundation springs at piers No. 1 and No. 2 were estimated to be about 4,200 kips/in. and 2,200 kips/in. per support, respectively. The vertical springs were assumed to be infinitely rigid. The south end supports were considered to be restrained in the vertical and transverse directions and free in the longitudinal direction.

The model was subjected to gravity and seismic loading. Seismic loading consists of two horizontal directions, using the 5% damped response spectrum (Type IV) with a PGA of 0.4g. Seismic loading in the vertical direction uses the same response spectrum with a scaling factor of 0.67. The seismic analysis in the vertical direction is not required by Caltrans criteria. However, it was considered in the analysis of this bridge because of a heavy CT.WT. mass attached to the superstructure.

The results from the response spectrum analysis in each of the three directions were combined on a square-root-of-the-sum-of-the-squares (SRSS) of the first 15 modes. The SRSS values obtained were then combined algebraically with dead loads according to the loading combinations described in Section 4.

The results of three-dimensional seismic analysis of the model with assumed foundation springs applied at the four pier supports are summarized as follows:

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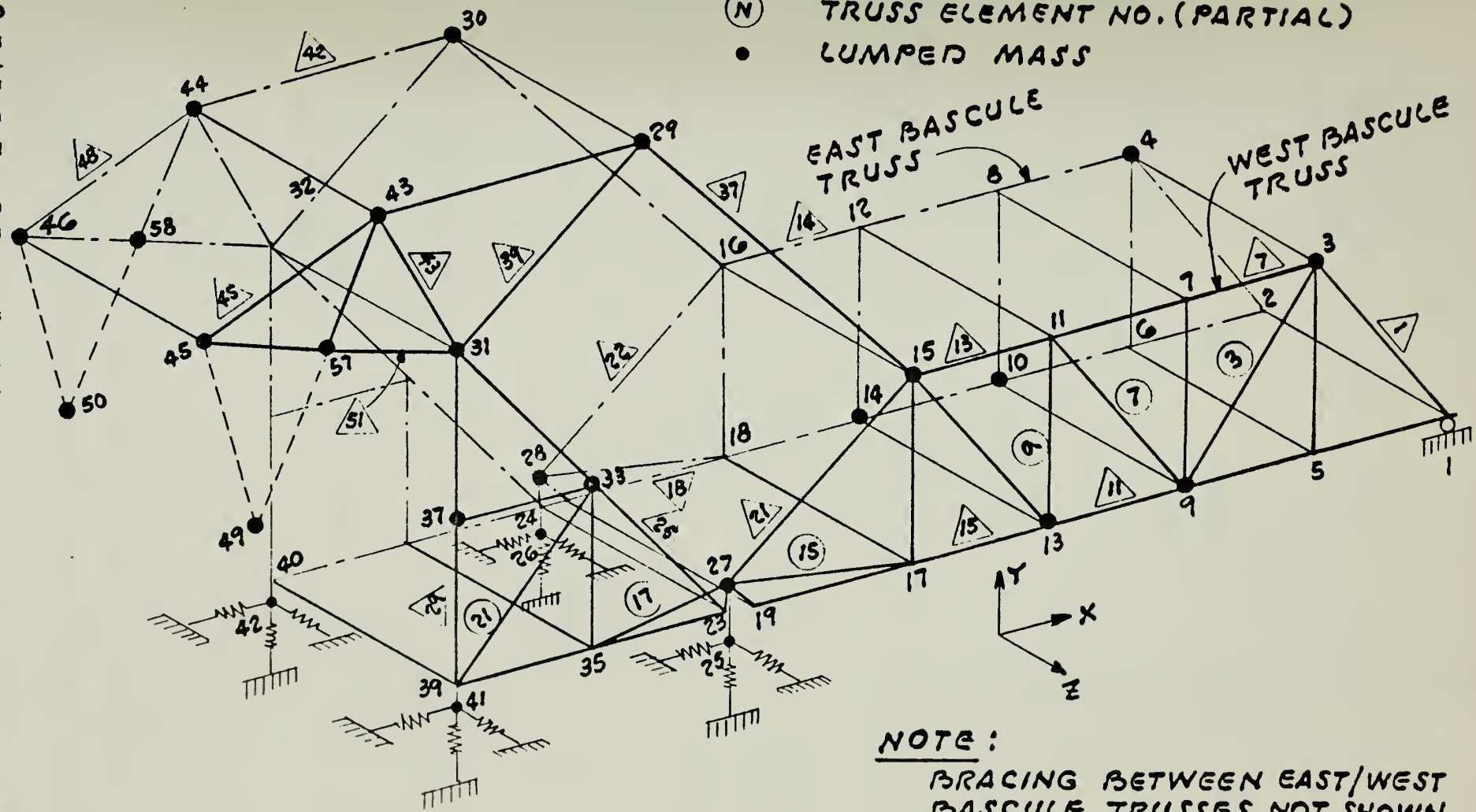

FIGURE 2 MATHEMATICAL MODEL - 3RD STREET BRIDGE OVER CHANNEL

1. The predominant periods of vibration in the three directions, along with their corresponding spectral acceleration (S_a) and participation factors, are given below:

	Mode	T (sec)	S_a (g)	Participation Factor
X-Direction (Longitudinal)	4	0.46	0.77	3.10
Y-Direction (Vertical)	2	0.68	0.48	1.38
Z-Direction (Transverse)	1	1.15	0.59	3.19

2. The deflections at various locations (see Figure 2) are listed below:

Nodal Point	δ_x (in.)	δ_y (in.)	δ_z (in.)
45	1.62	1.50	8.1
43	1.64	0.59	7.7
31	1.41	0.44	7.2
29	1.59	0.70	7.1
15	0.80	0.90	6.9
3	0.52	0.72	7.0

The maximum lateral deflection at the top of CT.WT. tower (i.e., Node 31) was calculated to be 7.2 in. in the transverse direction. This results in a drift ratio of 0.011 (7.2/666) which is within the allowable limit of 0.015 based on ATC-3 requirements for building design (Reference 8).

3. The combined ratios of the computed bending and axial stresses to allowable capacity values, set forth in Section 4, for critical beam and truss elements are presented in Tables 2 and 3. The results shown in the tables indicate that all structural members would meet the stress criteria with the exception of the following members:

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Results of Seismic Evaluation - 3rd Street
Bridge Over Channel (Truss Elements)

Member* I.D.	Nodal* Point	A (in^2)	F_a (Ksi)	Dead plus Seismic** f_a (Ksi)	Dead plus Seismic** $\frac{f_a}{1.7\ F_a}$	Section	
3	3 - 9	23.4	11.7	17.7	0.89	2-C15 x 40	
7	9 -11	31.4	18.0	16.5	0.58	4-L4 x 4 x 3/8, 2-PL 20 x 1/2	Bascule Truss
9	11-13	29.3	12.7	18.7	0.87	2-C15 x 50	Bascule Truss
15	17-27	70.9	13.8	13.5	0.58	4-L4 x 4 x 5/8, 2-PL 20 x 1 5/16	Bascule Truss
17	27-35	55.0	14.1	11.7	0.49	4-L4 x 4 x 1/2, 2-PL 20 x 1	CT WT Tower
21	33-39	31.4	13.2	7.0	0.31	Same as Member 7	CT WT Tower
25,26	1-6, 5-2	6.1	18.0	22.2	0.74	2-L5 x 3 1/2 x 3/8	Floor Bracing
29,30	9-14, 13-10	6.1	18.0	32.0	1.05 X	"	Floor Bracing
31,32	13-18, 17-14	6.1	18.0	25.5	0.85	"	Floor Bracing
33,34	17-20, 19-18	7.4	18.0	26.6	0.89	2-C12 x 25	Floor Bracing
43,44	11-16, 15-10	12.2	18.0	13.8	0.46	4-L5 x 3 1/2 x 3/8	Bascule Truss Top Bracing
45,46	15-30, 29-16	12.2	18.0	24.6	0.82	"	Bascule Truss Top Bracing
47,48	29-44, 43-30						
49,50	43-46, 45-44	11.4	18.0	20.0	0.68	4-L4 x 4 x 3/8	CTWT Truss Bracing
51,52	29-32, 31-30						
53,54	31-44, 43-42	12.2	18.0	15.9	0.53	4-L5 x 3 1/2 x 3/8	CTWT Truss Bracing
55~58	31-46, 45-32	19.0	18.0	27.6	0.92	4-L6 x 4 x 1/2	CTWT Truss Bracing
61,62	49-46, 45-50	12.2	18.0	16.3	0.54	4-L5 x 3 1/2 x 3/8	CTWT Truss Bracing
63,64	37-32, 31-38	26.5	12.5	19.6	0.93	2-L15 x 45 - CTWT Tower	

* See Figure 2 for member locations

** For the critical load combination

X Does not meet criteria

TABLE 2

Results of Seismic Evaluation - 3rd Street over Channel (Beam Elements)

Member* I.D.	Nodal* Point	Dead plus Seismic**				
		f_a (Ksi)	f_b (Ksi)	$\frac{f_a}{1.7F_a}$	$\frac{f_b}{1.7F_b}$	$\frac{f_a}{1.7F_a} + \frac{f_b}{1.7F_b}$
1	1- 3	7.2	13.4	0.24	0.45	0.69
7	3- 7	11.0	6.2	0.37	0.20	0.57
11	9-13	16.7	1.8	0.61	0.06	0.67
13	11-15	19.3	11.3	0.64	0.38	1.02 x
14	12-16	17.0	12.7	0.57	0.42	0.99
15	13-17	16.8	1.2	0.62	0.04	0.66
18	18-20	9.4	0.9	0.40	0.03	0.43
21	15-27	4.7	20.0	0.20	0.67	0.87
22	16-28	2.8	20.5	0.11	0.68	0.79
23	23-27	2.9	20.3	0.10	0.68	0.78
25	23-33	19.3	6.5	0.72	0.21	0.93
29	37-39	8.3	19.6	0.29	0.65	0.94
37	15-29	13.0	6.3	0.43	0.21	0.64
39	29-31	12.7	8.5	0.50	0.28	0.78
42	30-44	16.7	4.3	0.56	0.14	0.70
43	31-43	16.4	14.0	0.61	0.46	1.07 x
44	32-44	13.2	13.7	0.49	0.46	0.95
45	43-45	13.4	3.5	0.44	0.12	0.56
48	44-56	13.5	11.1	0.45	0.37	0.82
51	31-45	13.5	10.5	0.47	0.35	0.82

* See Figure 2 for member locations

** For the critical load combination

x Does not meet criteria

S.F. BRIDGE : GROUP V-1 TABLE 3

- Two floor bracing members in the middle of the movable span (Nodes 9 - 14 and 10 - 13)
- One member in the west bascule truss (Nodes 11 - 15)
- One member in the CT.WT. truss (Nodes 31 - 43)

However, the overstress ratios of the above members are in the range of 1.02 to 1.07. Thus, they may be considered to be acceptable.

4. All other members not shown in Tables 2 and 3 were found to be adequate.

It appears from the above analysis that all steel members would meet the performance criteria when the bridge in a closed position is subjected to the postulated earthquake motion. The lateral deflections or drift ratios at the top of CT.WT. tower are also acceptable.

Note that the results presented above are based on the computer model (see Figure 2) with the estimated springs applied at the four supports. However, a sensitivity study of the foundation springs assumed for the 4th Street Bridge indicates that there is virtually no difference in response properties between the two foundation conditions (i.e., rigid versus flexible). Therefore, it may be concluded that the variation in the foundation springs would not affect the reliability of the results calculated in this section.

5.2 Bridge in Open Condition:

Seismic structural analysis was not performed for the bridge in open condition. Reasons are as follows:

1. Gravity loading analysis performed for this bridge indicates that the bridge in closed position would be subjected to larger reaction forces at the trunnion supports (see Drawing No. B9581).

2. Based on the statistics of the bridge operation, this bridge is in closed position most of the time (estimated to be about 98%).

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3. Seismic analysis of the 3rd Street Bridge over Islais Creek indicates that the bridge in closed condition is more critical than in open position (see discussions on 3rd Street Bridge over Islais Creek presented in this report).

Based on the above evidence, it is considered unnecessary to perform seismic analysis for the bridge in open condition.

6. RECOMMENDATIONS AND COST ESTIMATES FOR REMEDIAL WORK

6.1 This bridge is in an area which has been previously identified as having liquefaction potential. Insufficient soil data is available at this time to confirm whether or not the potential actually exists at this specific site. We recommend that additional borings be made in the bridge approaches and that a grouting program be undertaken if necessary.

6.2 Estimated Cost:

Additional soil investigation	$5,000
Pressure grouting (if required)	15,000
Engineering supervision and inspection	3,000
Total	$23,000

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7. SUMMARY AND CONCLUSIONS

This bridge has been subjected to a rather detailed seismic structural analysis. The analysis shows that the bridge has sufficient lateral force-resisting capacity for the postulated earthquake. Therefore, strengthening is not required for this bridge.

Field inspection of the bridge reveals that the steel portion of the bridge is generally in good condition except that the lower portions of the girder near the bearing areas at the southeast corner of the movable deck are heavily corroded. The reinforced concrete south approach is in poor condition and in need of rehabilitation work.

According to available geotechnical data, the bridge site is underlain by artificial fill deposits. Soil liquefaction and lateral spreading are likely to occur during a major earthquake. However, the risk of landslide is minimum because the bridge lies on relatively flat ground. Cement and/or chemical grouting are recommended to increase its cohesive strength and thus reduce its liquefaction potential. The cost is not estimated because additional field exploration is required to define the extent of the loose material and its thickness.

FOURTH STREET BRIDGE OVER CHANNEL

V-2

CONTENTS

		page
1.	INTRODUCTION	V2-1
2.	SITE INVESTIGATION	V2-2
3.	BRIDGE DESCRIPTION	V2-4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE	V2-8
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE	V2-10
	5.1 Bridge in Closed Condition - Flexible Foundation	V2-10
	5.2 Bridge in Closed Condition - Rigid Foundation	V2-14
	5.3 Bridge in Open Condition	V2-15
6.	SEISMIC ANALYSIS OF THE MODIFIED BRIDGE	V2-16
	6.1 Modification Concepts	V2-16
	6.2 Analysis Results	V2-16
7.	COST ESTIMATE FOR STRUCTURAL MODIFICATIONS	V2-20
8.	SUMMARY AND CONCLUSIONS	V2-21
	REFERENCES	APPENDIX C

TABLES

1	Section Properties - Existing Structure	V2-6
2	Results of Seismic Evaluation - Existing Structure	V2-13
3	Results of Seismic Evaluation - Modified Structure	V2-19

FIGURES

1	Elevation and Plan	V2-5
2	Mathematical Model	V2-11
3	Structural Modifications	V2-17

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1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

2. SITE INVESTIGATION

Results of the site investigation for all bridges are given in Reference 3. A general summary of this reference as it pertains to this bridge under consideration is given in this section.

The bridge is located along the 4th Street crossing over Channel Street Waterway. Available geological information indicates that the site is underlain by artificial fill deposits (Qaf). The underlying bedrock is Franciscan formation of the Nob Hill belt. The depth to the bedrock is estimated to be approximately 150 ft. An exploratory boring, DH-10, was drilled at the northern approach of the bridge to explore the subsurface conditions. The boring was 30 ft deep and the materials encountered consisted of approximately 16 ft of artificial fill, loose to medium dense sand (SP) underlain by soft younger bay mud (CL) extending to the depth explored. Standard blow counts of the younger bay mud (CL) were 3 and 4 per foot of penetration. The younger bay mud had a torvane undrained shear strength of 450 psf. Ground water level was measured at a depth of 8 ft. The thickness of bay mud is estimated to be approximately 40 ft, based on other information.

The bridge is located at approximately 9 mi northeast of the San Andreas fault and 10 mi southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g and 0.3g, respectively. The maximum ground surface velocity is estimated to be approximately 15 in. per second., Response Spectrum III, shown on Figure 2 of Reference 3, and Response Spectrum IV, shown on Appendix B, reduced by 20%, are recommended for seismic safety evaluation.

The low density of the fill materials suggests that liquefaction is likely during a major earthquake. Although the bridge is pile supported, liquefaction of the fill material would result in some structural damage to the bridge. The traffic will be disrupted. Soil liquefaction may be alleviated by grouting of the loose sandy material. Cement and/or chemical grouting of the sandy material would increase its cohesive strength and thus reduce its liquefaction potential. Alternatively, well-drained stone columns or wick drains may be placed at about 5 ft center to center with a

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gravel drainage blanket beneath the pavement. The stone column and wick drain solutions may alleviate the soil liquefaction but result in several inches of settlement after a major earthquake. Specific recommendations for construction plans and specifications may be developed. Additional field exploration is required to define the extent of the loose material and its thickness.

The bridge lies on relatively flat ground, and the risk of landslide is minimal. Localized slope failure towards the channel, however, may occur during a major earthquake. The problem of slope instability may be alleviated by placement of rip-rap and filter materials.

The bridge is located outside the pre-1860 shoreline, as shown on Plate 3 (Reference 3), where soil lateral spreading may occur during a major earthquake. The risk of lateral spreading will be significantly reduced if the liquefaction potential is decreased by the above recommended solutions. Placement of rip-rap and proper filter materials would further protect the slope from failure into the channel

The bridge is located in the area of potential tsunami inundation, as shown on Plate 11 (Reference 3).

3. BRIDGE DESCRIPTION

This bridge is a single-leaf bascule steel structure with reinforced concrete slab approach spans. The structure was designed in 1915. It is 61 ft wide by 205 ft long and is oriented in the northerly direction. Its substructure consists of two abutments and two bridge piers, all on 18-in.-diameter vertical and battered wood piles. Elevation and plan of the bridge is illustrated in Figure 1.

The north and south approaches are made up of reinforced concrete slabs, beams, and columns. The columns are on 18 in. by 18 in. vertical concrete piles. The movable steel bascule truss can be rotated around the main trunnion with the help of a concrete counterweight (CT.WT.) of 1,408 kips. When the bridge is in a closed position, the south end of the bascule truss rests on the concrete pier. It has a restrainer to prevent movement in the transverse direction but allows free movement in the longitudinal direction.

The steel portion of the bridge includes two trusses spanned at 46 ft center to center, two main trunnion posts and a counterweight tower. There are two links and two support arms at the top and bottom of the counterweight, which are used to transfer the load to the tower and the trusses, respectively. The floor system consists of steel grating, I-section steel stringers, built-up steel floor beams, and six diagonal bracing members. All the steel members are made of built-up sections with riveted connections. Table 1 shows member sizes for representative steel members. No major structural alterations have been made to the bridge since its completion in 1917, except that in 1940 the diagonal bracing members of the bridge floor were reinforced and steel grating was installed to replace the wooden floor system.

When the bridge is subjected to an earthquake in the longitudinal direction, the two link members and two CT.WT. support arms would serve as connectors to transfer the lateral load to the CT.WT. tower and the trusses, respectively, and in turn to the bridge piers. In the transverse direction, the majority of the lateral force would be transferred to the trusses through the two CT.WT. arms. No effective connectors are provided at the

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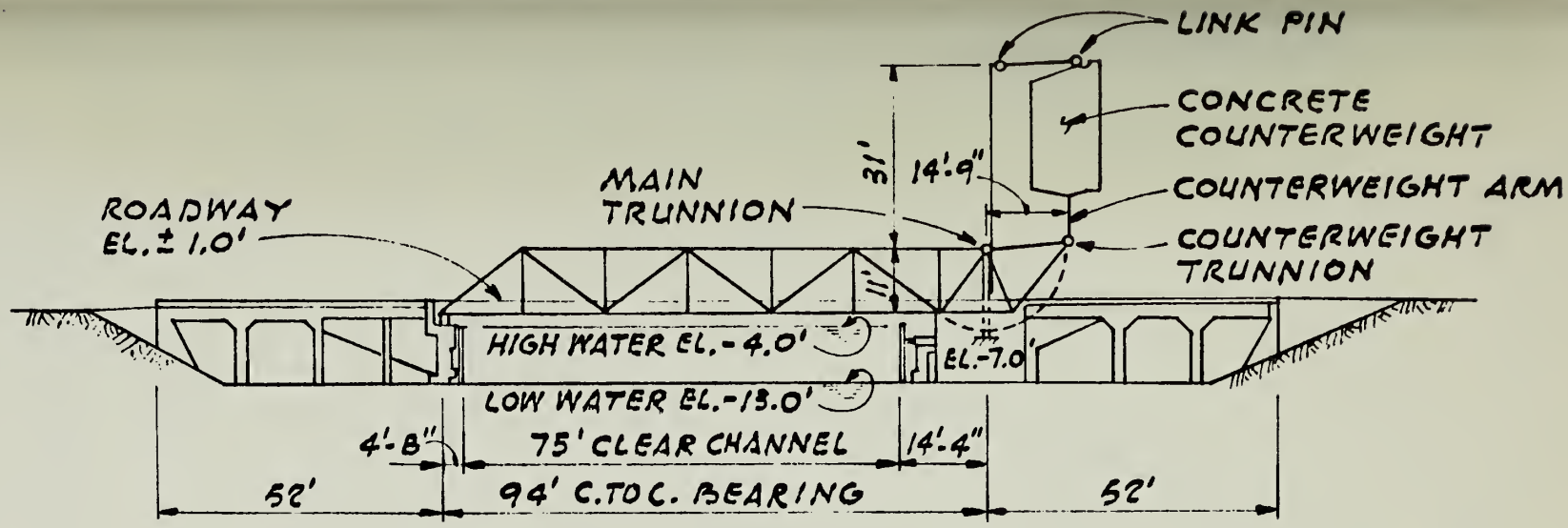
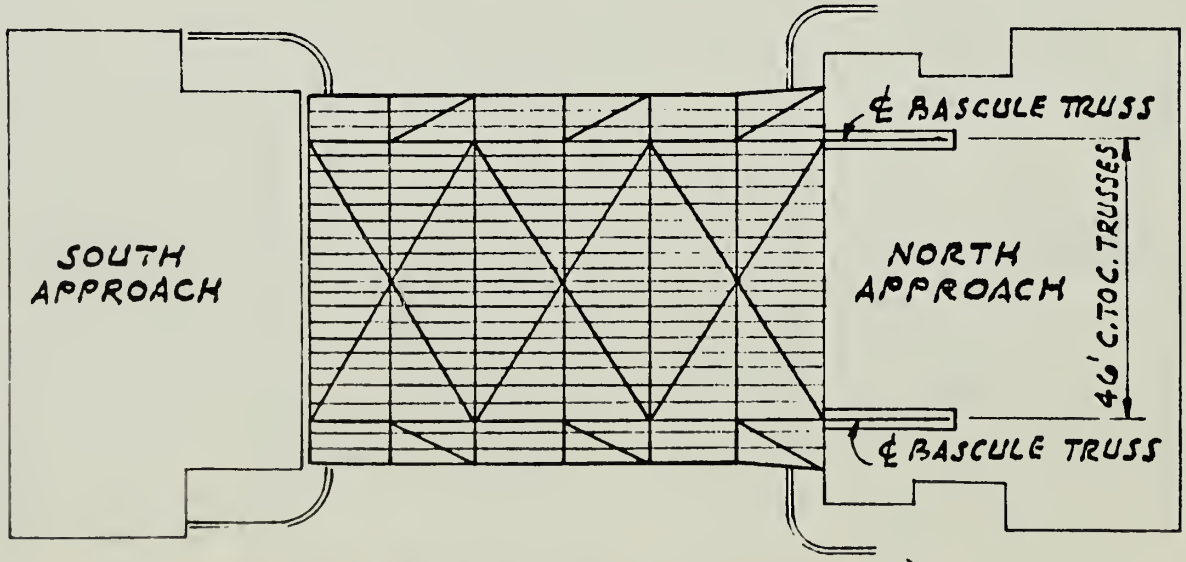


ELEVATION



PLAN OF FLOOR SYSTEM

ELEVATION AND PLAN OF 4TH ST. BRIDGE OVER CHANNEL

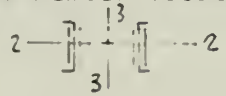
FIGURE 1

Member* I.D.	A (in^2)	S_{2-2} (in^3)	S_{3-3} (in^3)	F_a (Ksi)	F_b (Ksi)	Section
△1, △2	69.2			16.4	18.0	} See Figure 3
△3, △4	51.5			15.0	↑	
△5, △6	51.5	84	530	15.0		
△7, △8	51.5	84	530	15.0		
△46, △47	26.5	100	349	15.9		2-C15 x 45
△50, △51	30.9	109	355	18.0		2-C15 x 35 w/2-3/8" PLS
△58, △59	59.5	109	683	18.0		2-C15 x 50 w/2-5/8" & 2-1/2" PLS
△60, △61	76.7	190	813	18.0		2-C15 x 50 w/2-3/4" & 2-13/16" PLS
△62, △63	58.9	166	677	16.3		2-C15 x 50 w/2-1/2" & 2-5/8" PLS
△70, △71	38.0	127	437	16.3		2-C15 x 40 w/2-1/2" PLS
△74, △75	40.8	129	463	16.3		2-C15 x 50 w/2-3/4" PLS
△76, △77	58.9	166	677	16.1	18.0	2-C15 x 50 w/2-1/2" & 2-5/8" PLS
①, ②, ③, ⑥	2.86	-	-	18.0		L 4 x 4 x 3/8 (Floor Bracing)
㉔, ㉕	20.0	-	-	14.8		2-C14 x 33 - Link
㉖	4.2	-	-	18.0		2-L 4 x 3 x 5/16 - CTWT Tower Bracing
㉟, ㊱	24.7	-	-	16.3		4-L 6 x 4 x 7/16, 1 PL-21 x 3/8"
㊶, ㊷	41.5	-	-	16.3		4-L 8 x 6 x 5/8, 1 PL-21 x 3/8"
㊺, ㊻	76.7	-	-	18.0		Same as △60 & △61

See Typ. Section below**

*See Figure 2 for member locations

**Typical section



S.F. BRIDGE - GROUP V-2

TABLE 1

top of CT.WT. to transfer lateral force to the CT.WT. tower. The lateral force-resisting system of the floor deck consists of diagonal bracing to resist lateral forces in both the longitudinal and transverse directions.

Field inspection of the bridge reveals that the steel portion of the bridge on the road and above is generally in good condition with the exception of the two trunnion posts and some other members which show signs of built-up rust at several locations. No apparent structural deficiencies regarding riveted joints and CT.WT. connections were observed. However, the bridge may be in need of repainting. It is suggested that the new painting be undertaken along with the structural modifications to be recommended later in Section 6. Note that the structural conditions under the bridge floor, as well as the substructures, were not subjected to field investigation in this study. However, based on the field inspection report conducted by Caltrans in 1982, the following structural deficiencies were found:

1. The reinforced concrete approaches are in very poor condition. Most of the haunches between the horizontal beams and the columns have large spalls with exposed reinforcement. Some of the columns have large cracks just below the haunches.

2. Some of the floor beams have built-up rust between the web and the bottom flange angles.

The average daily traffic flow over the bridge was estimated to be approximately 6,500, based on the available information published by Caltrans in 1982. As for the traffic flow under the bridge, an interview with the bridge operator indicates that the bridge is lifted about 2 or 3 times every day and about 4 to 5 minutes during each operation.

Note that some discrepancies were found from a comparison of the drawings and the as-built conditions of the trunnion and tower posts. According to the drawings, the trunnion and tower posts were designed per Drawing No. 1376, dated May 13, 1916, and revised on July 21, 1916. However, neither the original design nor the revised scheme is in agreement with the as-built conditions which will be shown in Figure 3.

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4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE

The criteria for analysis of the existing building, as well as for the modified building, are given in Appendix A. A general summary of the appendix as it pertains to the bridge discussed here is given below:

1. Because the bridge is located in an area with soft bay mud, a Caltrans response spectrum IV, shown in Appendix B, reduced by 20%, will be used for seismic safety evaluation. The peak ground acceleration (PGA) of the response spectrum is 0.4g, with a damping of 5% critical.

2. The existing and modified structures will be checked for the following four loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

3: $U = D.L. + (E.Q.X + E.Q.Y)/Z$

4: $U = D.L. + (E.Q.Z + E.Q.Y)/Z$

where

U	=	Total load to be resisted
D.L.	=	Dead load
E.Q.X	=	Earthquake load in X-direction (longitudinal)
E.Q.Y	=	Earthquake load in Y-direction (vertical)
E.Q.Z	=	Earthquake load in Z-direction (transverse)
Z	=	Adjustment factor for ductility and risk

3. The adjustment factor Z for ductility and risk for structural steel members and connections are as follows:

Member	Z
Beam	3
Top & Bottom Chords of Truss	3
Column & Bracing	2
Connection	1.5

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4. Allowable stresses for both new and old construction will be taken as follows:

Members	1.7 F_s
Connections	1.3 F_s

The term F_s for the existing steel members is the allowable working stress based on F_y = 30 ksi. Similarly, F_s for new steel members is the allowable working stress based on F_y = 36 ksi. Both levels of working stresses are given in the AISC Manual of Steel Construction, eighth edition.

5. Except as noted in the drawings, the connections will be assumed to be 7/8-in.-diameter rivets with an allowable shear capacity of 10 kips for each rivet.

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5. SEISMIC ANALYSIS OF THE EXISTING BRIDGE

5.1 Bridge in Closed Condition - Flexible Foundation:

Linear and three-dimensional finite element response spectrum analyses were undertaken using the SAP IV computer program (Reference 4). Figure 2 represents the superstructure of the bridge. The model consists of 79 beam elements and 63 truss elements with nodal masses applied at appropriate nodes. The foundation springs simulating the pile and pier foundations were modeled at the supporting piers as shown in the figure. The pile foundation springs were calculated according to the procedure suggested in Reference 6. The foundation springs were estimated to be about 2,200 kips/in. per support in each of the two principal horizontal directions (i.e., longitudinal and transverse). The vertical springs were assumed to be infinitely rigid. Since the foundation stiffness values were estimates only, a sensitivity study of the springs applied will be discussed in Section 5.2. The south end supports were considered to be restrained in the vertical and transverse directions and free in the longitudinal direction.

The model was subjected to gravity and seismic loading. Seismic loading consists of two horizontal directions, using the 5% damped response spectrum (Type IV) with a PGA of 0.4g. Seismic loading in the vertical direction uses the same response spectrum with a scaling factor of 0.67. The seismic analysis in the vertical direction is not required by Caltrans criteria. However, it was considered in the analysis of this bridge because of a heavy CT.WT. mass attached to the superstructure.

The results from the response spectrum analysis in each of the three directions were combined on a square-root-of-the-sum-of-the-squares (SRSS) of the first ten modes. The SRSS values obtained were then combined algebraically with dead loads according to the loading combinations described in Section 4.

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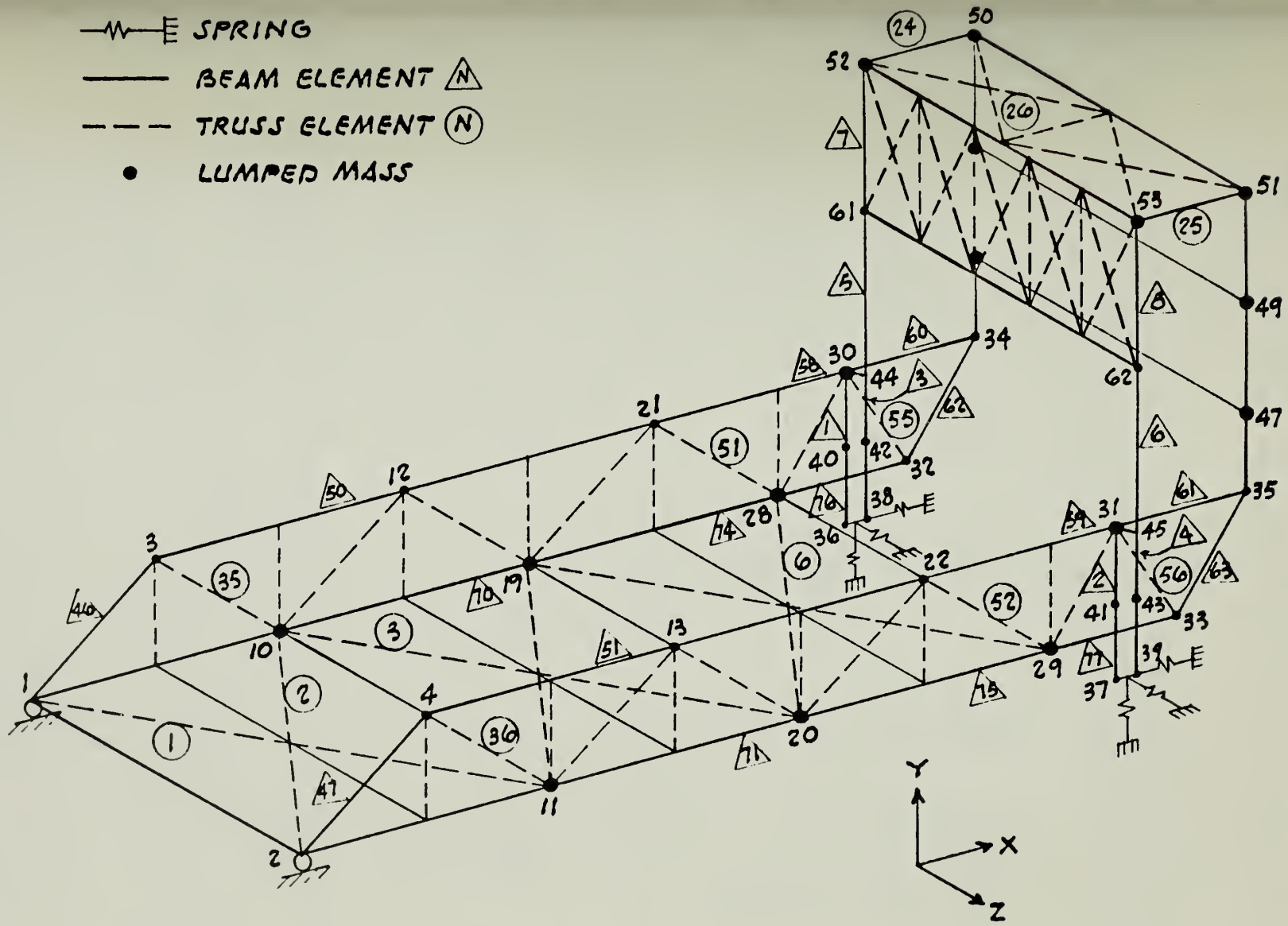

MATHEMATICAL MODEL

4TH ST. BRIDGE OVER CHANNEL

FIGURE 2

The results of three-dimensional seismic analysis of the model with assumed foundation springs applied at the two pier supports are summarized as follows:

1. The predominant periods of vibration in the three directions, along with their corresponding spectral accelerations (S_a) and participation factors, are given below:

	Mode	T (sec)	S_a (g)	Participation Factor
X-Direction (Longitudinal)	1	1.74	0.49	1.84
	5	0.34	0.72	0.76
Y-Direction (Vertical)	4	0.35	0.73	1.00
	9	0.19	0.61	1.76
Z-Direction (Transverse)	2	0.90	0.65	2.05
	3	0.49	0.78	0.79

2. The deflections at various locations (see Figure 2) are listed below:

Nodal Point	δ_x (in.)	δ_y (in.)	δ_z (in.)
52	19.7	0.06	5.6
34	0.62	0.72	2.7
30	0.61	0.17	0.94

The maximum lateral deflections at the top of CT.WT. tower was calculated to be 19.7 in. and 5.6 in. in the longitudinal and transverse directions, respectively. This results in a drift ratio of 0.039 (19.7/504) which greatly exceeds the allowable limit of 0.015 based on ATC-3 requirements for building design (Reference 8).

3. The combined ratios of the computed bending and axial stresses to allowable capacity values, set forth in Section 4, for critical beam and truss elements are presented in Table 2. The results shown in the table indicate that the CT.WT. posts (Elements 5

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Results of Seismic Evaluation - 4th Street Over Channel (Existing)

Member* I.D.	Dead Load plus Seismic**					Note
	f_a (Ksi)	f_b (Ksi)	$\frac{f_a}{1.7F_a}$	$\frac{f_b}{1.7F_b}$	$\frac{f_a}{1.7F_a} + \frac{f_b}{1.7F_b}$	
(1), (2)	18.2	42.0	0.65	1.41	2.05	X
(3), (4)	14.1	21.3	0.52	0.71	1.23	X
(5), (6)	0.64	80.0	0.03	2.66	2.69	X
(7), (8)	~	35.0	~	1.17	1.17	X
(46), (47)	3.70	1.35	0.39	0.05	0.44	
(50), (51)	8.20	3.20	0.27	0.11	0.38	
(58), (59)	18.2	3.6	0.61	0.12	0.73	
(60), (61)	8.0	33.9	0.27	1.13	1.40	X
(62), (63)	18.9	27.9	0.68	0.93	1.61	X
(70), (71)	9.5	1.35	0.34	0.05	0.39	
(74), (75)	18.1	7.48	0.65	0.25	0.90	
(76), (77)	20.7	5.1	0.76	0.17	0.93	
(1)	39.4	-	1.31	-	1.31	X
(2)	37.0	-	1.24	-	1.24	X
(3)	24.1	-	0.80	-	0.80	
(6)	22.4	-	0.75	-	0.75	
(24), (25)	5.4	-	0.21	-	0.21	
(26)	4.9	-	0.16	-	0.16	
(35), (36)	7.4	-	0.36	-	0.36	
(51), (52)	9.3	-	0.45	-	0.45	
(55), (56)	16.2	-	0.54	-	0.54	

* See Figure 2 for member locations

** For the critical load combination

X Does not meet criteria

S.F. BRIDGE : GROUP V-2 TABLE 2

through 8) and members connecting to the CT.WT. support arms (Elements 60 through 63) exceed the capacity limits set forth in this report. A few diagonal members near the south end of the floor deck were also found to be overstressed (Elements 1 and 2).

4. All other members not shown in Table 2 were found to be adequate.

It appears from the above analysis that some steel members would be overstressed when the bridge is subjected to the postulated earthquake motion. The lateral deflections or drift ratios at the top of CT.WT. tower are also not acceptable.

5.2 Bridge in Closed Condition - Rigid Foundation:

The results presented in Section 5.1 are based on the computer model (see Figure 2) with the estimated foundation springs applied at the two supports. To study the sensitivity of the springs assumed, a computer analysis was performed assuming that the foundations are rigid, in lieu of flexible as in the previous analysis. The dynamic analysis in the longitudinal direction was conducted and a comparison of dynamic responses for both cases are summarized as follows.

Foundation	T (sec)	Participation Factor	δ_x (in.)
Flexible	1.76	1.84	19.7
	0.34	0.76	0.3
Rigid	1.75	1.83	19.6
	0.34	0.74	0.3

A comparison of periods and lateral deflections at the top of CT.WT. tower under the two foundation conditions indicates that there is virtually no difference in response properties between the two cases. Therefore, it may be concluded that the variation in the foundation springs would not affect the reliability of the results calculated in the previous section.

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5.3 Bridge in Open Condition:

Seismic structural analysis was not performed for the bridge in open condition. Reasons are as follows:

1. Based on the statistics of the bridge operation, this bridge is in closed position most of the time (estimated to be about 99%).

2. Seismic analysis of the 3rd Street Bridge over Islais Creek indicates that the bridge in closed condition is more critical than in open position (see discussions on 3rd Street Bridge over Islais Creek presented in this report).

3. Gravity loading analysis performed for the bridge located at 3rd Street over Channel Street Waterway also indicates that the bridge in closed position would be subjected to larger reaction forces at the trunnion supports (see Drawing No. B9581).

Based on the above evidence, it is considered unnecessary to perform seismic analysis for the bridge in open condition.

6. SEISMIC ANALYSIS OF THE MODIFIED BRIDGE

6.1 Modification Concepts:

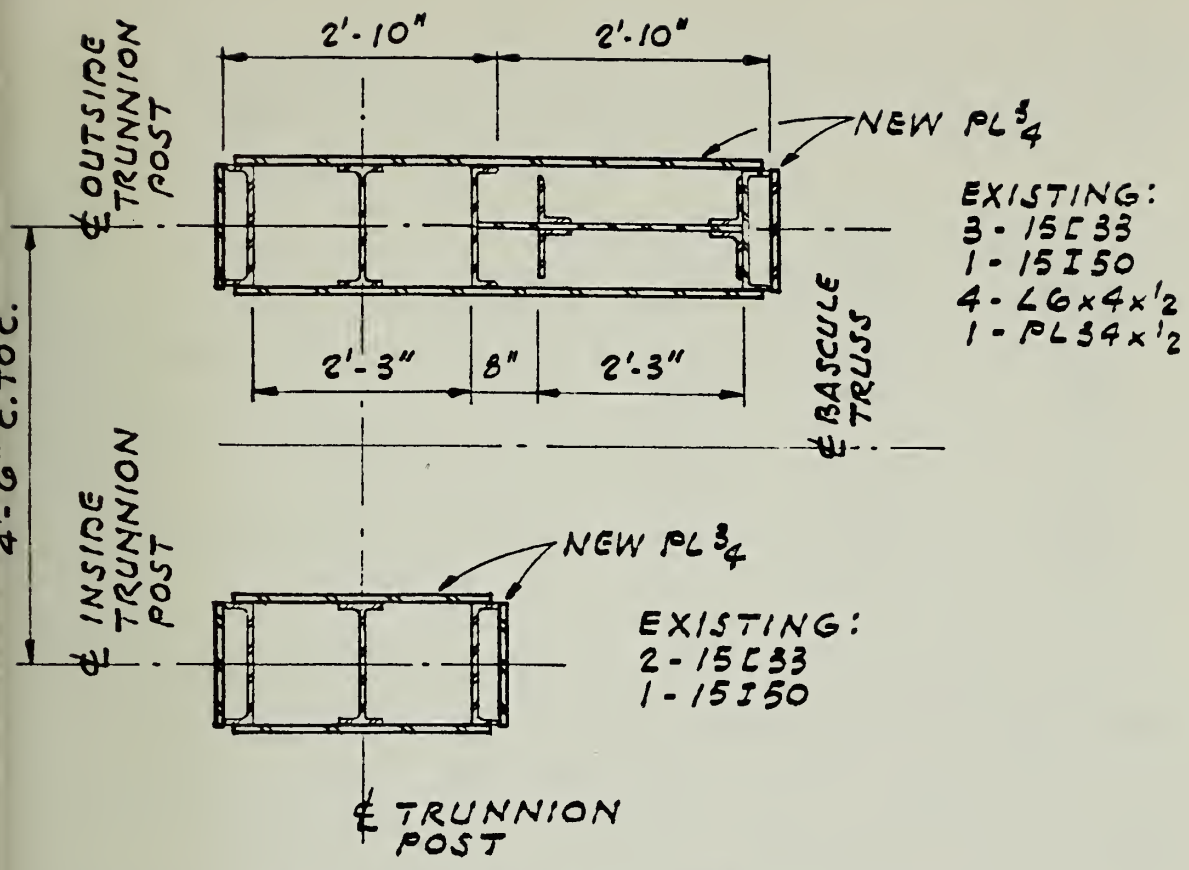
The results of detailed seismic structural analysis of the bridge in closed condition indicate that the CT.WT. tower columns as well as the main trunnion columns would exceed the capacity criteria set forth in Section 4. The calculated lateral deflection of 19.7 in. at the top of the CT.WT. tower greatly exceeds the allowable drift limit. In addition, some diagonal members in the floor and truss chord members connecting the CT.WT. support arms were found to be overstressed. Therefore, strengthening is needed to increase the lateral load capacity of major structural elements and to prevent the failure of the CT.WT. supporting members.

There are several ways to strengthen the bridge; however, the proposed modification schemes to be discussed in this chapeter are considered to be the best alternative. It will cause a minimum amount of disruption to the traffic flow, both under and above the bridge. In addition, it is cost-effective.

The proposed modification schemes are illustrated in Figure 3. The schemes call for the addition of structural steel plates to strengthen the existing member sections. The existing diagonal lace members would be removed and new steel elements would be welded to the existing steel numbers. In addition, the overstressed diagonal members in the bridge floor would be strengthened by adding a new angle (L4 x 4 x 3/8) to the existing diagonal members of similar size.

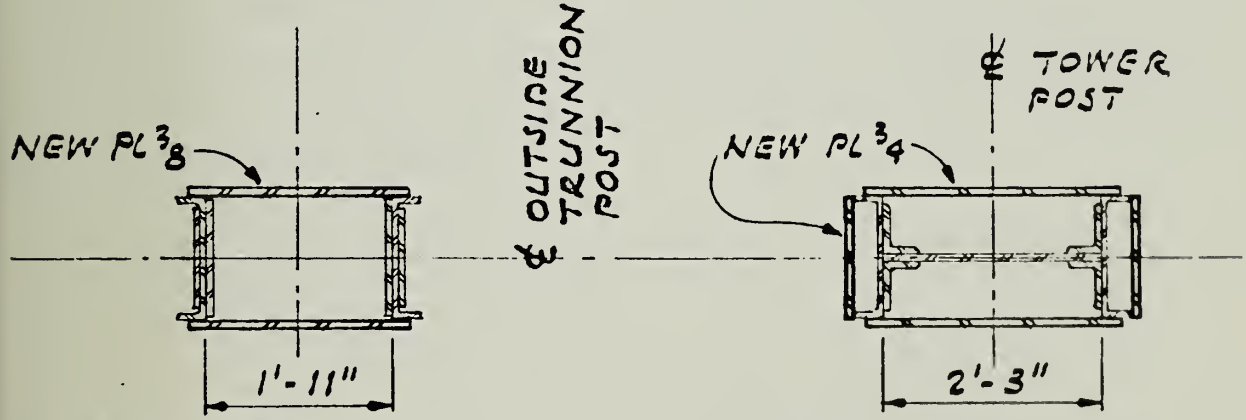
6.2 Analysis Results:

The modified bridge was subjected to three-dimensionally dynamic analysis using the same model (Figure 2), except that some member sizes were changed to reflect the proposed modification concepts illustrated in Figure 3. The resulting longitudinal, vertical, and transverse seismic response analyses are summarized in the following.



TOWER SECTION BELOW TRUNNION

(MEMBERS 1 THRU 4)



SECTION OF MEMBERS

(MEMBERS G2 & G3)

TOWER SECTION ABOVE TRUNNION

(MEMBERS 5 THRU 8)

STRUCTURAL MODIFICATIONS

F. BRIDGE - GROUP V-2

FIGURE 3

1. The predominant periods of vibration in the three directions, along with their corresponding spectral accelerations (S_a) and participation factors, are given below:

	Mode	T (sec)	S_a (g)	Participation Factor
X-Direction	1	1.06	0.67	1.89
	5	0.33	0.72	0.65
Y-Direction	4	0.35	0.73	1.13
	9	0.18	0.61	1.77
Z-Direction	2	0.73	0.71	2.05
	3	0.48	0.77	0.78

2. The maximum lateral deflection at the top of CT.WT. tower was calculated to be 8.76 in., in comparison with 19.7 in. before the modification. The drift ratio was found to be 0.017, which is slightly above the ATC-3 recommended limit of 0.015 for building structures. However, it is considered that a drift of 0.017 would be acceptable for the bridge structure in this report.

3. The results of stress analysis for the proposed modified bridge are shown in Table 3. The results show that all members presented in the table are below the stress ratio of 1.0 with the exception of two members. However, the overstressed ratio of 1.05 would be considered acceptable.

4. All other members not shown in Table 3 were also found to be adequate.

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Results of Seismic Evaluation - 4th Street Over Channel

(Modified Structure)

Member[a] I.D.	A ($in.^2$)	S_{2-2} ($in.^3$)	S_{3-3} ($in.^3$)	Dead Load Plus Seismic[d]					
				f_a (ksi)	f_b (ksi)	$\frac{f_a}{1.7F_a}$	$\frac{f_b}{1.7F_b}$	$\frac{f_a}{1.7F_a}$	$\frac{f_b}{1.7F_b}$
1, 3[b]	195.2[c]	1012	2714	11.9	22.2	0.42	0.61	1.03	
5	150.0[c]	-	1426	-	38.0	-	1.05	1.05	
7	150.0[c]	-	1426	-	16.1	-	0.45	0.45	
60	76.7	-	813	7.7	21.4	0.26	0.71	0.97	
62	76.2[c]	295	-	14.2	13.0	0.51	0.36	0.87	
74	40.8	-	463	17.5	7.3	0.63	0.24	0.87	
76	58.9	-	677	20.0	5.0	0.73	0.17	0.95	
(1)	5.72[c]	-	-	18.5	-	0.62	-	0.62	
(2)	5.72[c]	-	-	17.3	-	0.58	-	0.58	
(3)	2.86	-	-	20.6	-	0.69	-	0.69	
(24)	20.0	-	-	7.2	-	0.29	-	0.29	
(26)	4.2	-	-	17.5	-	0.58	-	0.58	

. See Figure 2 for member locations

. These two members are treated as a composite section, see Figure 3

. These members are to be strengthened

. For the critical load combination

F. BRIDGE : GROUP V-2 TABLE 5

7. COST ESTIMATE FOR STRUCTURAL MODIFICATIONS

7.1 The preliminary cost estimate for the proposed strengthening is given below:

Description	Quantity	Unit	Total
Demolition	--	LS	$ 5,000
Structural Steel	39,000	LB	39,000
Welding (3/8" thick)	1,000	LF	20,000
Miscellaneous	--	LS	5,000
Subtotal			$69,000
Contractor's OH&P (20%)			13,800
Contingency (20%)			16,560
Total			$99,360
Engineering			20,000
Escalation (8%)			8,000
	TOTAL		$127,360

Note that the above estimated cost does not include work on rehabilitation of the concrete approaches recommended by Caltrans in its 1982 field inspection report (see Section 3) and the new painting on the metal surfaces recommended in this study.

7.2 This bridge is in an area which has been previously identified as having liquefaction potential. Insufficient soil data is available at this time to confirm whether or not the potential actually exists at this specific site. We recommend that additional borings be made in the bridge approaches and that a grouting program be undertaken if necessary.

7.3 Estimated Cost of Liquefaction Protection:

Additional soil investigation	$5,000
Pressure grouting (if required)	15,000
Engineering supervision and inspection	3,000
Total	$23,000

8. SUMMARY AND CONCLUSIONS

The results of detailed seismic structural analysis of the bridge indicate that the counterweight (CT.WT.) tower columns as well as the main trunnion columns would exceed the capacity criteria set forth in this study report. The calculated lateral deflection of 19.7 in. at the top of the CT.WT. tower greatly exceeds the allowable drift limit. In addition, some diagonal members in the bridge floor and truss chord members connecting the CT.WT. support arms were found to be overstressed. Therefore, strengthening is needed to increase the lateral load capacity of major structural elements and to prevent the failure of the CT.WT. supporting members. The cost is estimated at about $127,000.

Field inspection of the bridge reveals that the steel portion of the bridge on the road and above is generally in good condition with the exception of the two posts below the main trunnion and some other members which show signs of built-up rust at several locations. However, the reinforced concrete approaches are in very poor condition and in need of complete rehabilitation work. It is also suggested that the new painting on metal surfaces be undertaken along with the structural modifications.

According to available geotechnical data, the bridge site is underlain by artificial fill deposits. Soil liquefaction and lateral spreading are likely to occur during a major earthquake. However, the risk of landslide is minimum because the bridge lies on relatively flat ground. Cement and/or chemical grouting are recommended to increase its cohesive strength and thus reduce its liquefaction potential.

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THIRD STREET BRIDGE OVER ISLAIS CREEK

V-3

THIRD STREET BRIDGE OVER ISLAIS CREEK

CONTENTS

		page
1.	INTRODUCTION	V3-1
2.	SITE INVESTIGATION	V3-2
3.	BRIDGE DESCRIPTION	V3-4
4.	CRITERIA FOR ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE	V3-8
5.	SEISMIC ANALYSIS OF THE EXISTING BRIDGE	V3-10
	5.1 Bridge in Closed Condition	V3-10
	5.2 Bridge in Open Condition	V3-14
6.	RECOMMENDATIONS AND COST ESTIMATES FOR REMEDIAL WORK	V3-16
7.	SUMMARY AND CONCLUSIONS	V3-17
	REFERENCES	APPENDIX C

TABLES

1	Section Properties - Existing Structure	V3-6
2	Results of Seismic Evaluation - Closed Condition	V3-13
3	Comparison of Member Forces for the Bridge in Closed and Open Conditions	V3-15

FIGURES

1	Elevation and Plan	V3-5
2	Mathematical Model	V3-11

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1. INTRODUCTION

Historically, bridges in California have been designed with little or no consideration given to seismic design. Where seismic forces were considered in design, they were usually based on the lateral force recommendations of the Structural Engineers Association of California.

Bridge failures resulting from the San Fernando earthquake of 1971 gave evidence to the fact that bridges are unique structures and that seismic provisions written for buildings are not always appropriate for bridges. Since that time, the American Association of State Highway and Transportation Officials (AASHTO) has amended the Standard Specification for Highway Bridges (Reference 1) to include seismic design provisions, and the California Department of Transportation has developed some special requirements (Reference 2).

The City and County of San Francisco has undertaken a program to upgrade the seismic resistance of existing bridges owned by the City and County. As part of the program, the City and County of San Francisco authorized URS/John A. Blume & Associates, Engineers, under Department of Public Works Order #132,428, to perform a seismic capacity investigation of nine concrete and three steel bridges. The purpose of the investigation is to determine whether or not remedial work is required to bring the bridges up to current seismic safety standards and, if remedial work is required, to recommend a remedial program and estimate the cost.

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2. SITE INVESTIGATION

Results of the site investigation for all bridges are given in Reference 3. A general summary of this reference as it pertains to this bridge under consideration is given in this section.

The bridge is located along 3rd Street over Islais Creek. Available geological information indicates that the site is underlain by artificial fill deposits (Qaf). The underlying bedrock is Franciscan Formation of the Fort Point-Hunters Point shear zone. The depth to the bedrock is estimated to be approximately 100 ft. An exploratory boring, DH-11, was drilled at the northern approach of the bridge to explore the subsurface conditions. The boring was 40 ft deep and the materials encountered consisted of approximately 10 ft of artificial fill, medium dense gravelly sand (SW), underlain by soft to medium stiff clayey silt (ML) with lenses of silty clay (CL) extending to the depth explored. Standard blow counts of the upper gravelly sand (SW) were 15 and 18 per foot of penetration. Standard blow counts of the silty sand (ML) ranged from 8 to 63 per foot of penetration. Ground water level was measured at a depth of 11 ft. The thickness of the bay mud is estimated to be 40 ft, based on other information.

The bridge is located approximately 8 mi northeast of the San Andreas fault and 11 mi southwest of the Hayward fault. The maximum bedrock and ground surface accelerations for the maximum credible earthquake are estimated to be 0.5g and 0.3g, respectively. The maximum ground surface velocity is estimated to be approximately 15 in. per second. Response Spectrum III, shown on Figure 2 of Reference 3, and Response Spectrum IV, shown in Appendix B, reduced by 20%, are recommended for the seismic safety evaluation.

Plate 9 of Reference 3 shows that the bridge is located in an area of potential liquefaction. The medium dense fill materials and portions of the clayey silt may liquefy during a major earthquake. Although the bridge is pile supported, liquefaction of the fill material would result in some structural damage of the bridge. The traffic will be disrupted. Soil liquefaction may be alleviated by grouting of the loose sandy material. Cement and/or chemical grouting of the sandy material would increase its

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cohesive strength and thus reduce its liquefaction potential. Alternatively, well-drained stone columns or wick drains may be placed at about 5 ft center to center, with a gravel drainage blanket beneath the pavement. The stone column and wick drain solutions may alleviate the soil liquefaction but result in several inches of settlement after a major earthquake. Additional field exploration is required to define the extent of the loose material and its thickness.

The bridge lies on relatively flat ground, and the risk associated with landslide is minimal. The site is located in the pre-1860 shoreline area, as shown on Plate 3 (Reference 3), where soil lateral spreading may occur during a major earthquake. The risk of lateral spreading will be significantly reduced if the liquefaction potential is decreased by the above recommendations.

The bridge is located in the area of potential tsunami inundation, as shown on Plate 11 (Reference 3).

3. BRIDGE DESCRIPTION

This bridge is a double-leaf bascule steel structure with reinforced concrete slab approaches. The structure was designed in 1945. The bridge deck is supported by two concrete box-type structures and is oriented in a northerly direction. The foundations are supported by 18-in. by 18-in. vertical and battered concrete piles. Rip-rap slope protection is supported by wood piles. The bridge has a width of 100 ft and a maximum clear span of 97 ft between fenders. Elevation and plan of the bridge are illustrated in Figure 1.

The movable steel structure consists of 6 box-type steel girders, wide flange steel beams and stringers, and diagonal bracing members. The steel deck can be rotated around the main trunnions with the help of a counterweight (CT.WT). The trunnion supports and CT.WT. are situated inside the concrete structure. There are 6 trunnions (3 on each side) supported by wide flange steel columns (W14 x 264 for exterior and W14 x 356 for interior). These steel columns are anchored to the concrete walls and extended to the bottom slab of the concrete box-type structure. Unlike the other two bascule bridges over Channel Street, there are no restrainers in the free ends. There is a 4-in. gap between two free ends. Table 1 shows member sizes for representative steel members.

The lateral force-resisting system of the floor deck in both the longitudinal and transverse directions is made up of diagonal bracing to resist lateral forces which, in turn, are carried to the trunnion supports. When the bridge is subjected to an earthquake in the vertical direction, the floor deck would act like a cantilever supported around the trunnion. There are bearing blocks and shock absorbers at both ends of CT.WT. to resist vertical impact to the concrete structure.

No structural alteration has been made to the bridge since its completion. The left girder (G6 as indicated in Figure 1) is currently under repair. According to the field inspection report by Caltrans in 1981, this girder was hit by a ship or boat, causing damage to the bottom flange angles and web plate. Additional damage to one of the right girders (G1) was noted

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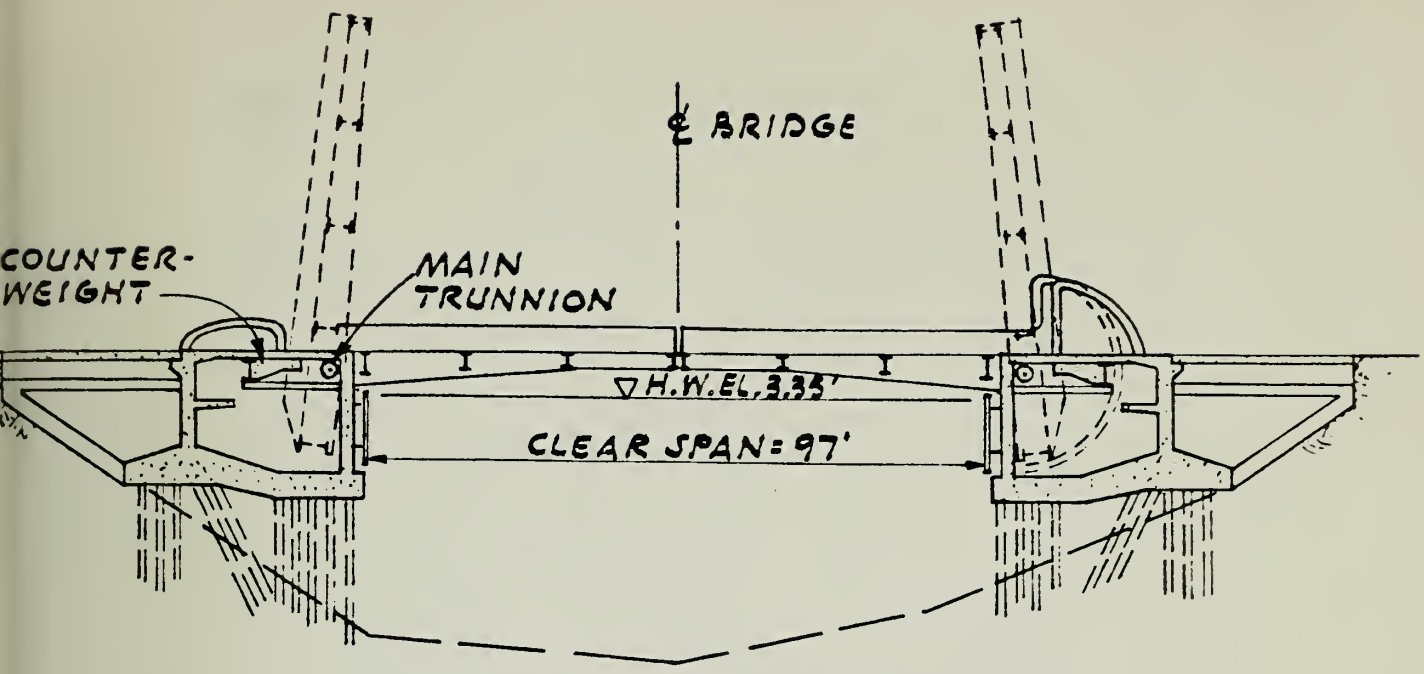
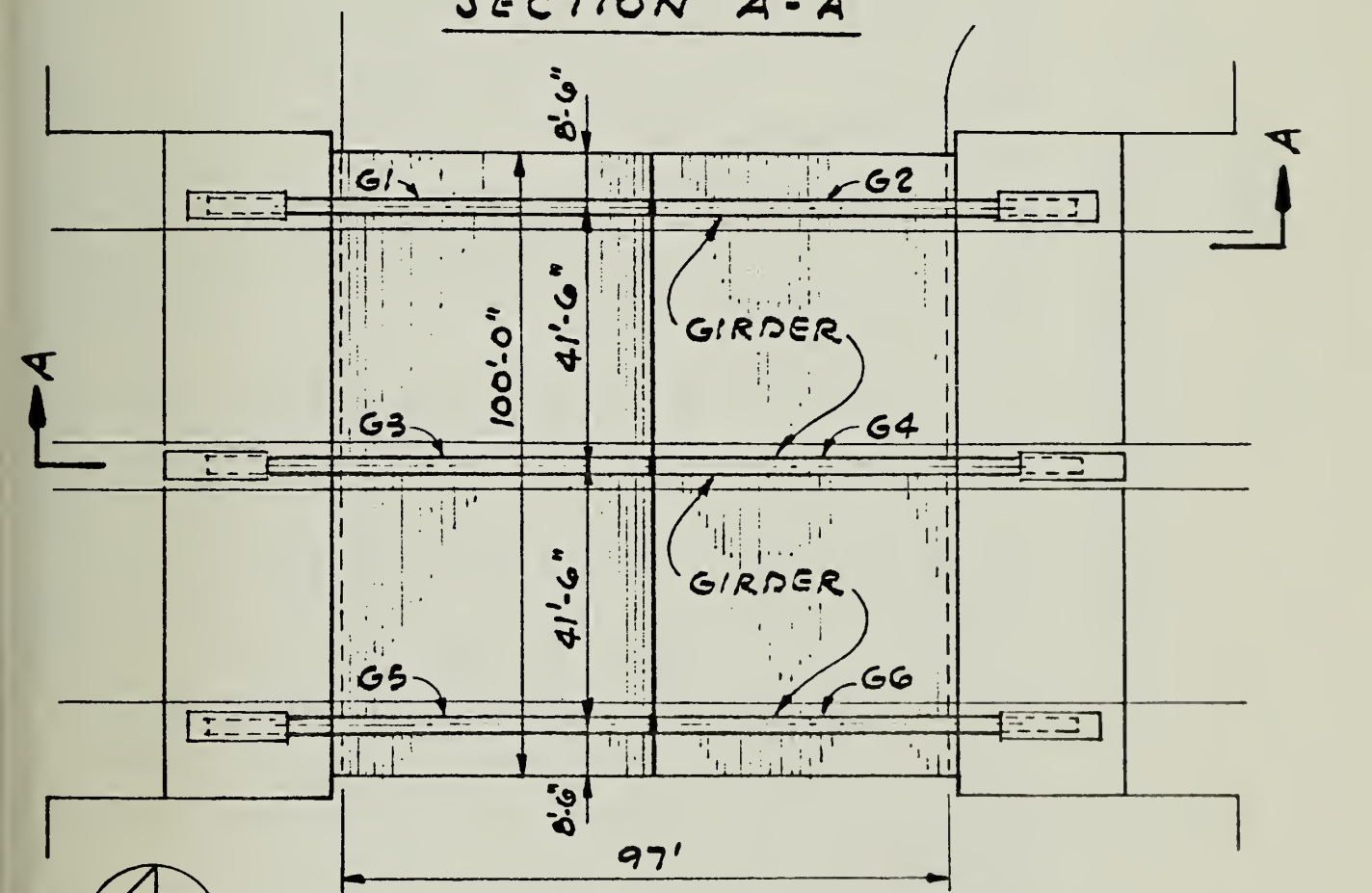


SECTION A-A



PLAN

ELEVATION AND PLAN OF

3RD ST. BRIDGE OVER ISLAIS CREEK

'.F. BRIDGE : GROUP V-3 FIGURE 1

Section Properties - 3rd Street Bridge over Islais Creek

Member* I.D.	Nodal* Points	A ($in.^2$)	S_{2_2} ($in.^3$)	S_{3_3} ($in.^3$)	Section **
1, 3	1-4, 3-6	125	2760	1510	4-L 8 x 6 x 1/2, 2 -1/2 WEB, 2-3/8 T&B
2	2 - 5	144	3422	1718	4-L 8 x 6 x 7/8, 2-1/2 WEB, 2 - 3/8 T&B
4, 6	4-11, 6-13	130	3208	1584	Same as members 1 & 3, except height
5	5 - 12	152	3833	1905	4-L 8 x 6 x 1, 2-1/2 WEB, 2-3/8 T&B
7, 9	11-18,13-20	165	5317	2024	4-L 8 x 8 x 7/8, 2-1/2 WEB, 2-3/8 T&B
8	12 - 19	217	6019	2757	2-L 8x6x1, 4-L 8x4x1, 2-1/2 WEB, 2-3/4 T&B
10, 12	18-21,20-23	229	7298	3064	2-L 8x8x7/8, 4-L 8x4x1, 2-1/2 WEB 2-3/4 T&B
11	19 - 22	240	8206	3101	2-L 8x6x1, 4-L 8x4x1, 2-1/2 WEB, 2-3/4 T&B
13, 15	21-27,23-29	229	7298	3064	Same as members 10 & 12 (assumed)
14	22 - 28	240	8206	3101	Same as member 11 (assumed)
25, 26 39, 40	1 -2-3 18-19-20	44.8	487	47	W 33 x 152
27 ~ 32 33 ~ 38	4-5-6 11-12-13	57.2	669	62	W 36 x 194
(1) ~ (4)	-	2.13	-	-	C 4 x 7.25
(5) ~ (8)	-	2.86	-	-	L 5 x 3 x 3/8
(9) ~ (12)	-	3.61	-	-	L 6 x 4 x 3/8

* See Figure 2 for member locations

** Typical girder Section

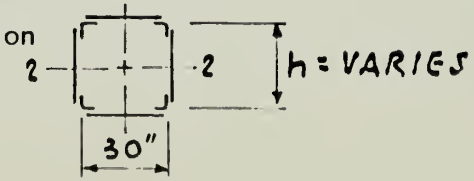


TABLE 1

from our field investigation. The bottom web plate of the girder was damaged and pushed inward about 1 in., causing a dish effect. It was also hit by a boat and in need of repair. Aside from the structural damage mentioned above, the bridge is generally in good condition. No apparent structural deficiencies regarding riveted joints and CT.WT. connections were observed.

The average daily traffic flow over the bridge was estimated to be approximately 20,000, based on the available information published by Caltrans in 1981. As for the traffic flow under the bridge, an interview with the bridge operator indicates that the bridge is lifted about 20 times per month and 10 to 30 minutes during each operation.

4. CRITERIA FOR SEISMIC ANALYSIS OF THE EXISTING AND MODIFIED BRIDGE

The criteria for analysis of the existing structure, as well as for the modified structure, are given in Appendix A. A general summary of the appendix as it pertains to the bridge discussed here is given below:

1. Because the bridge is located in an area with soft bay mud, a Caltrans response spectrum IV, shown in Appendix B, reduced by 20%, will be used for seismic safety evaluation. The peak ground acceleration (PGA) of the response spectrum is 0.4g, with a damping of 5% critical.

2. The existing and modified structures will be checked for the following four loading combinations:

Load Case 1: $U = D.L. + (E.Q.X + 0.3\ E.Q.Z)/Z$

2: $U = D.L. + (E.Q.Z + 0.3\ E.Q.X)/Z$

3: $U = D.L. + (E.Q.X + E.Q.Y)/Z$

4: $U = D.L. + (E.Q.Z + E.Q.Y)/Z$

where

U	=	Total load to be resisted
D.L.	=	Dead load
E.Q.X	=	Earthquake load in X-direction (longitudinal)
E.Q.Y	=	Earthquake load in Y-direction (vertical)
E.Q.Z	=	Earthquake load in Z-direction (transverse)
Z	=	Adjustment factor for ductility and risk

3. The adjustment factor Z for ductility and risk for structural steel members and connections are as follows:

Member	Z
Beam	3
Top & Bottom Chords of Truss	3
Column & Bracing	2
Connection	1.5

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4. Allowable stresses for both new and old construction will be taken as follows:

Members	1.7 F_s
Connections	1.3 F_s

The term F_s for the existing steel members is the allowable working stress based on F_y = 30 ksi. Similarly, F_s for new steel members is the allowable working stress based on F_y = 36 ksi. Both levels of working stresses are given in the AISC Manual of Steel Construction, eighth edition.

5. Except as noted in the drawings, the connections will be assumed to be 7/8-in.-diameter rivets with an allowable shear capacity of 10 kips for each rivet.

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5. SEISMIC ANALYSIS OF THE EXISTING BRIDGE

5.1 Bridge in Closed Condition:

Linear and three-dimensional finite element response spectrum analyses were undertaken using the SAP IV computer program (Reference 4). Figure 2 represents the superstructure of the bridge. The model consists of 46 beam elements and 12 truss elements with nodal masses applied at appropriate nodes. The foundation springs simulating the pile and trunnion supports were modeled at the supporting piers as shown in the figure. The pile foundation springs were calculated according to the procedure suggested in Reference 6. The horizontal foundation springs were estimated to be about 1,400 kips/in. and 1,600 kips/in. per support in the longitudinal and transverse directions, respectively. The vertical springs were assumed to be infinitely rigid. The south end supports were considered to be free in all directions.

The model was subjected to gravity and seismic loading. Seismic loading consists of two horizontal directions, using the 5% damped response spectrum (Type IV) with a PGA of 0.4g. Seismic loading in the vertical direction uses the same response spectrum with a scaling factor of 0.67. The seismic analysis in the vertical direction is not required by Caltrans criteria. However, it was considered in the analysis of this bridge because of the cantilever action in this direction.

The results from the response spectrum analysis in each of the three directions were combined on a square-root-of-the-sum-of-the-squares (SRSS) of the first 14 modes. The SRSS values obtained were then combined algebraically with dead loads according to the loading combinations described in Section 4.

The results of three-dimensional seismic analysis of the model with assumed foundation springs applied at the three trunnion supports are summarized as follows:

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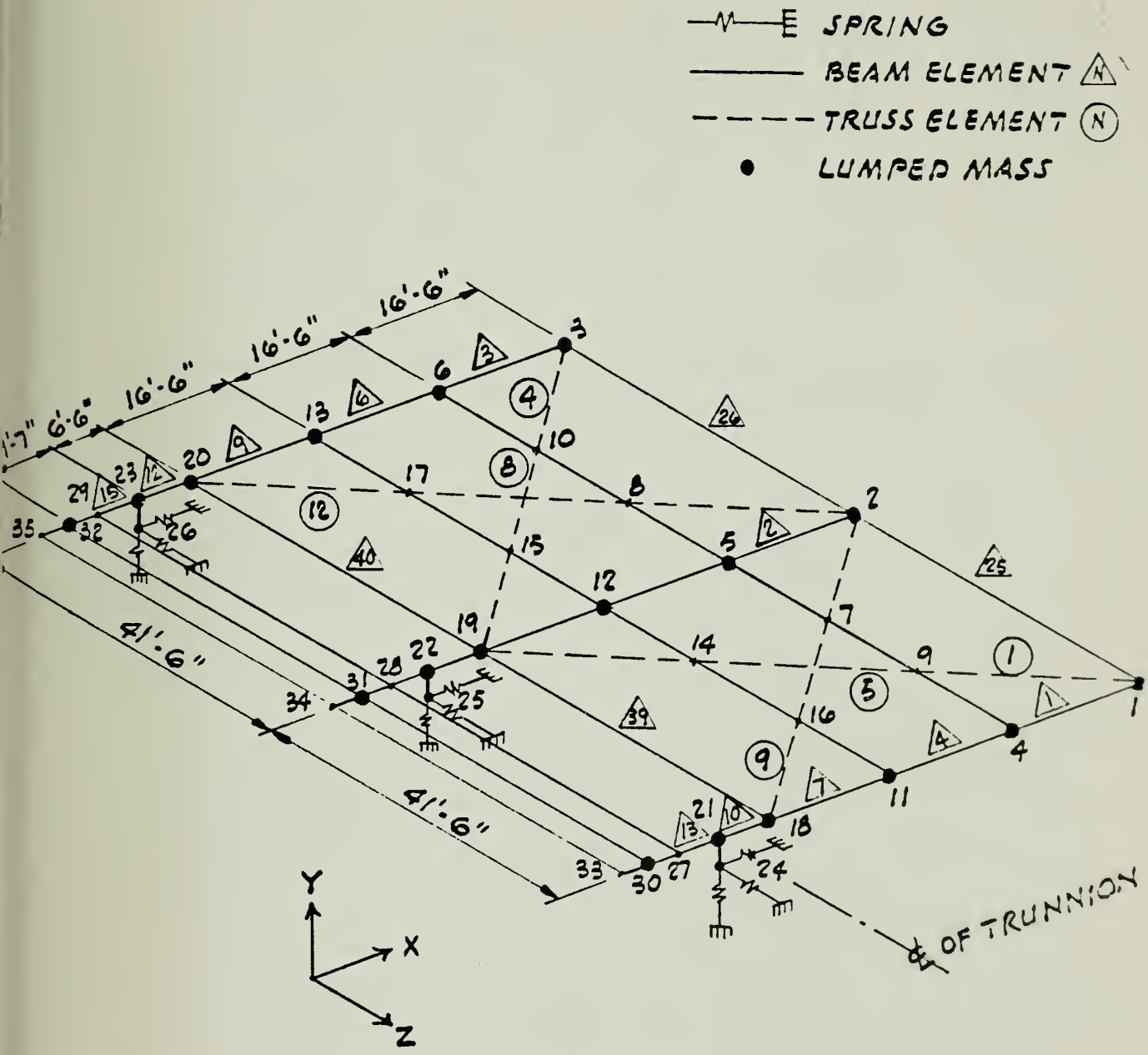

MATHEMATICAL MODEL

3RD ST. OVER ISLAIS CREEK

S.F. BRIDGE : GROUP V-3 FIGURE 2

1. The predominant periods of vibration in the three directions, along with their corresponding spectral acceleration (S_a) and participation factors, are given below:

	Mode	T (sec)	S_a (g)	Participation Factor
X-Direction (Longitudinal)	1	0.86	0.64	1.34
	5	0.24	0.65	2.20
Y-Direction (Vertical)	8	0.13	0.39	1.78
	10	0.07	0.34	0.58
Z-Direction (Transverse)	2	0.34	0.72	1.00
	6	0.23	0.65	1.78

2. The deflections at the free and CT.WT. due to dead plus seismic loads in the three principal directions are listed below:

Location	δ_x (in.)	δ_y (in.)	δ_z (in.)
Free End	0.41	2.1 down 0.1 up	1.1
CT.WT.	0.32	0.23 down 0.21 up	0.26

The maximum deflection at the free end was calculated to be 0.41 in. in the longitudinal direction, which is less than the allowable limit of 2 in. provided between the two free ends of the bridge deck. The maximum calculated vertical deflection at the free end was 2.1 in. This results in a drift ratio of 0.003 (2.1/672) which is within the allowable limit of 0.015 based on ATC-3 requirements for building designs (Reference 8). The calculated deflections at the CT.WT. were in the range of 0.2 to 0.3 in.

3. The combined ratios of the computed bending and axial stresses to allowable capacity values, set forth in Section 4, for critical beam and truss elements are presented in Table 2. The results

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Results of Seismic Evaluation - 3rd Street Bridge
Over Islais Creek (Closed Condition)

Member* I.D.	Element Type	F_a (Ksi)	F_b (Ksi)	Dead plus Seismic **		
				f_a (Ksi)	f_b (Ksi)	$\frac{f_a}{1.7F_a} + \frac{f_b}{1.7F_b}$
1, 3	Girder	-	18.0	-	2.1	0.07
4, 6		-		-	8.9	0.29
5		-		-	7.9	0.26
8		-		-	10.5	0.34
11		-		-	9.9	0.32
13, 15		-		-	7.5	0.25
14	Girder	-	18.0	-	9.4	0.31
①~④	Bracing	18.0	-	23.0	-	0.75
⑤~⑧	"	18.0	-	20.9	-	0.70
⑨~⑫	"	18.0	-	16.6	-	0.55

* See Figure 2 for member locations

** For the critical load combination

S.F. BRIDGE : GROUP V-3 TABLE 2

shown in the table indicates that all structural members would meet the specified stress criteria.

4. All other members not shown in Table 2 were found to be adequate.

It appears from the above analysis that all steel members would meet the performance criteria when the bridge in a closed position is subjected to the postulated earthquake motion. The lateral deflections calculated are also acceptable.

Note that the results presented above are based on the computer model (see Figure 2) with the estimated springs applied at the three supports. However, a sensitivity study of the foundation springs assumed for the 4th Street Bridge indicates that there is virtually no difference in response properties between the two foundation conditions (i.e., rigid versus flexible). Therefore, it may be concluded that the foundation springs assumed in the flexible model would not affect the reliability of the results calculated in this section

5.2 Bridge in Open Condition:

Despite the fact that this bridge is in closed position most of the time (estimated to be about 98%), the seismic structural analysis of the bridge in open condition was also performed in this study. A comparison of selected member forces (axial, shear, and moment) for the bridge in closed and open conditions is shown in Table 3. The results indicate that the bridge in closed position is more critical than in open position. Thus, it may be concluded that this bridge in open position would also satisfy the specified performance criteria.

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Comparison of Member Forces for the
Bridge in Closed and Open Conditions

Member* I.D.	Element Type	Dead Load plus Seismic**									
		Bridge in Closed Condition					Bridge in Open Condition (60°)				
		P (Kips)	V_{2-2} (Kips)	V_{3-3} (Kips)	M_{2-2} (Kips)	M_{3-3} (Kips)	P (Kips)	V_{2-2} (Kips)	V_{3-3} (Kips)	M_{2-2} (K-in.)	M_{3-3} (K-in.)
1, 3	Girder	-	-	-	5,918	-	-	-	-	4,467	-
4, 6	↑	-	-	-	28,560	-	-	-	-	14,610	-
5	↕	-	-	-	30,300	-	-	-	-	18,290	-
8	↕	-	-	-	64,000	-	-	-	-	27,640	-
11	↕	-	-	-	81,000	-	-	-	-	68,980	-
13, 15	↓	-	-	-	52,200	-	-	-	-	37,530	-
14	Girder	-	-	-	77,370	-	-	-	-	47,730	-
22, 24	Trunnion Support	854	144	251	-	-	861	125	239	-	-
23		950	143	211	-	-	834	126	216	-	-
①~④	Bracing	49	-	-	-	-	51	-	-	-	-
⑤~⑧	"	56	-	-	-	-	57	-	-	-	-
⑨~⑫	"	60	-	-	-	-	61	-	-	-	-

* See Figure 2 for member locations

** For the critical load combination.

TABLE 3

6. RECOMMENDATIONS AND COST ESTIMATES FOR REMEDIAL WORK

6.1 This bridge is in an area which has been previously identified as having liquefaction potential. Insufficient soil data is available at this time to confirm whether or not the potential actually exists at this site. We recommend that additional borings be made in the bridge approaches and that a grouting program be undertaken if necessary.

6.2 Estimated Cost:

Additional soil investigation	$5,000
Pressure grouting (if required)	15,000
Engineering supervision and inspection	3,000
Total	$23,000

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7. SUMMARY AND CONCLUSIONS

This bridge has been subjected to a rather detailed seismic structural analysis. The analysis shows that the bridge has sufficient lateral force-resisting capacity for the postulated earthquake. Therefore, strengthening is not required for this bridge.

Field inspection of the bridge reveals that the steel portion of the bridge is generally in good condition except that one of the right (easterly) girders was damaged by a ship. The bottom web plate of the girder was pushed inward about 1 in., causing a dish effect. This girder is in need of repair.

According to available geotechnical data, the bridge site is underlain by artificial fill deposits. Soil liquefaction and lateral spreading are likely to occur during a major earthquake. However, the risk of landslide is minimum because the bridge lies on relatively flat ground. Cement and/or chemical grouting are recommended to increase its cohesive strength and thus reduce its liquefaction potential. The cost is not estimated because additional field exploration is required to define the extent of the loose material and its thickness.

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APPENDIX A
CRITERIA

EVALUATION CRITERIA

CONTENTS

		page
1.	INTRODUCTION	A-1
2.	SEISMIC INPUTS	A-2
	2.1 Caltrans Response Spectra	A-2
	2.2 Adjustment Factors For Ductility and Risk	A-3
3.	MATERIAL PROPERTIES	A-4
4.	ANALYSIS PROCEDURES AND CRITERIA	A-5
	4.1 Analysis Procedures	A-5
	4.2 Orthogonal Effects	A-5
	4.3 Loading Combination	A-6
5.	DETERMINATION OF DEMAND/CAPACITY RATIO FOR SELECTED BRIDGE COMPONENTS	A-7
	5.1 Reinforced Concrete Bridges	A-7
	5.2 Movable Steel Bridges	A-8
6.	DEVELOPMENT OF SEISMIC RETROFITTING	A-10
	REFERENCES	APPENDIX C

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EVALUATION CRITERIA

1. INTRODUCTION

URS/John A. Blume & Associates, Engineers (URS/Blume), was authorized by the City of San Francisco to perform seismic structural analyses and prepare modification concepts for a total of 12 existing bridges in two groups (IIA and V). The objective of this effort is to investigate the ability of the bridges to safely resist seismic forces and to provide the methods and costs to strengthen the bridges to accomplish the objective.

This document was developed in accordance with Caltrans criteria (References 2 and 10). It describes the analysis criteria for the evaluation of the seismic analyses and the development of the remedial concepts for structural modifications.

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2. SEISMIC INPUTS

2.1 Caltrans Response Spectra (A•R•S):

All bridges will be analyzed for the site-specific response spectra for an earthquake that would result from a maximum credible earthquake along the San Andreas fault. The bridges under investigation, along with the estimated maximum bedrock accelerations and the recommended response spectra, are as follows:

Bridges	Maximum Bedrock Acceleration (g)		Response Spectrum (See Fig. 3a, Ref. 3)
IIA-1	0.5		II
IIA-2	0.5		II
IIA-3	0.5		II
IIA-4	0.5		II
IIA-5	0.5		II
IIA-6	0.5		II
IIA-7	0.5		I
IIA-8	0.5		I
IIA-9	0.5		II
V-1	0.5		IV
V-2	0.5	20%	IV
V-3	0.5	reduction	IV

It is noted that Bridges IIA-1 through IIA-6 are located in an area with depth of bedrock within 30 ft, Bridges IIA-7 and IIA-8 are located in an area with bedrock outcrops, Bridge IIA-9 is located in an area with approximately 40 ft of dense Colma Formation sand over bedrock, and Bridges V-1 through V-3 are located in an area with soft bay mud. A 20% reduction of the response spectra accelerations is recommended for three steel bridges located in the bay mud area. The above information and criteria were provided by Allstate Geotechnical Services (Reference 3).

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2.2 Adjustment Factor Z for Ductility and Risk:

Because of the large seismic forces involved in earthquakes and the infrequency of their occurrence, some allowance for inelastic action may be permitted. Therefore, seismic demand forces for individual members and connections of the bridge will be determined by dividing the elastic forces from the analysis (see Section 4) by the appropriate adjustment factor (Z).

Figure 1.2.20 of AASHTO (Reference 2) provides guidelines for determining the adjustment factor (Z) as a function of a structure's period of vibration. The following proposed factors are slightly different from those recommended by Caltrans criteria. The difference is because most existing column piers have deficiencies in transverse reinforcement and are not well confined. Smaller adjustment factors are therefore proposed for the analysis of the existing columns. In addition, the adjustment factors for structural steel members and connections are also presented specifically for the three movable steel bridges.

Component	Z Factor
Concrete Structure:	
Multiple-column pier (ductile)	8*
Multiple-column pier (bent)	3*
Single-column pier	2*
Wall-type pier	2*
Abutment Connection	0.8
Expansion joint	0.8
Column pier connections (to superstructure or foundation)	1.0
Steel Structure:	
Beam	3
Column and Bracing	2
Connection	1.5

*The Z-factor is to be used for modifying the bending moment of the column. For shear and axial forces, a Z-factor of 1.0 shall be used.

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3. MATERIAL PROPERTIES

For the determination of the strength and stiffness of a bridge, the material properties as provided by structural drawings will be used. However, in the absence of information, the following material specifications will be assumed:

Structural Steel:	f_y = 30 ksi
Reinforced Concrete:	f'_c = 2,500 psi
	f_y = 40 ksi

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4. ANALYSIS PROCEDURES AND CRITERIA

4.1 Spectral Analysis Procedures:

4.1.1 Single-mode Spectral Method. For bridges with well-balanced span and supporting bents or piers of approximately equal stiffness and mass, a single-mode spectral method recommended in the ATC-6 Guidelines (Reference 7) will be used. The method can be performed with manual calculation or with the aid of a computer program, such as SEISAB (Reference 5).

For single-span bridges, a detailed seismic analysis may not be required. However, the connection between the bridge span and the abutment will be evaluated, both longitudinally and transversely, to resist the gravity reaction forces at the abutment multiplied by the peak A•R•S coefficient of the site.

4.1.2 Response Spectrum Method. For relatively complex bridges (unbalanced stiffness, non-straight alignment, skews, and intermediate expansion joints), the dynamic analysis using the response spectrum modal superposition method shall be used. The computer program SEISAB will be used to perform such analysis. The total elastic demand forces (shear, moment, axial force, etc.) and displacement for the structure and components will be obtained by using the complete quadratic combination (CQC) method of the response of all significant modes.

For three movable steel bridges, computer program SAP IV (Reference 4) will be used for response analysis.

4.2 Orthogonal Seismic Effects:

A combination of orthogonal seismic forces will be used to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. The elastic seismic member forces and moments resulting

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from analysis in the two perpendicular directions of Section 4.1 will be combined to form two load cases as follows:

EQ Load Case 1: 100% of the seismic forces and moments for the longitudinal direction plus 30% of the seismic forces and moments for the transverse direction.

EQ Load Case 2: 100% of the seismic forces and moments for the transverse direction plus 30% of the seismic forces and moments for the longitudinal direction.

4.3 Loading Combination:

Seismic member forces and moments for the bridge components will be determined by dividing the elastic seismic forces and moments obtained from Load Case 1 and Load Case 2 of Section 4.2 by the appropriate adjustment factor Z of Section 2.2. The modified seismic forces and moments resulting from the two load cases will then be combined independently with forces and moments from other loads as specified in the following loading combination for the components.

$$U = 1.0\ (D + B + SF + E + EQ)$$

where:

U = total load to be resisted

D = dead load

B = buoyancy

SF = stream-flow pressure

E = earth pressure

EQ = elastic seismic load for either Load Case 1 or Load Case 2 of Section 4.2 modified by dividing by the appropriate Z-factor

Note that the above equation is consistent with AASHTO Group VII loading combination with $\gamma = 1.0$ and $\beta = 1.0$.

5. DETERMINATION OF DEMAND/CAPACITY RATIO FOR SELECTED BRIDGE COMPONENTS

5.1 Reinforced Concrete Bridges:

This section describes the methods for determining demand/capacity ratio for the components most likely to be damaged during an earthquake. These include:

- Reinforced concrete column piers
- Expansion joints and bearings
- Abutments
- Hold-down devices at supports

Foundation failures resulting from liquefaction of the underlying soil and/or excess earth movement have been the cause of the majority of bridge failures in some areas during past earthquakes. This subject will be discussed in Reference 3.

5.1.1 Column Piers. Demand forces and moments for columns and their connections at the top and bottom of the columns will be calculated based on the elastic demands determined in Section 4.3. An adjustment factor Z will be applied as given in Section 2.2. If the calculated demands exceed the ultimate capacity values estimate in accordance with strength reduction factors given in Section 1.5.30 of AASHTO, a further calculation may be required, i.e., evaluation of forces resulting from plastic hinging in columns.

The forces resulting from plastic hinging at the top and/or bottom of the column may be estimated in accordance with the procedures given in Section 4.8.2 of the ATC-6 Guidelines or Section 1.2.20(G) of AASHTO. The values corresponding to column hinging are generally smaller than those elastic demand forces and will be calculated only if the ratio of elastic demand/ capacity exceeds 1.0.

5.1.2 Expansion Joints and Bearings. Demand/capacity ratios for bridge expansion joints and bearings will be calculated for

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force in bearings which are restrained against movement or for displacement at unrestrained expansion joints.

When restrainers such as shear keys or anchor bolts are provided at the joint to resist seismic forces, shear demands will be determined from the elastic demand forces divided by a reduction factor of 0.8.

For unrestrained joints, the seat width provided at the bearings or expansion joints will be compared with the minimum value specified in Section 4.9 of the ATC-6 Guidelines. Alternatively, the relative displacement demands between different segments of the bridge may be calculated from the elastic spectral analysis of the bridge and will be compared with the allowable expansion provided at the joint.

5.1.3 Abutments. The primary consequence of the abutment failure is the loss of accessibility. These failures are caused by excessive lateral displacement of the bridge and/or settlement of approach fills. Therefore, demand/capacity ratios for displacement at the abutment will be evaluated. The displacement demand can be determined from the response analysis which considers the flexibility of the abutment while the displacement capacity is limited to 6 in.

The components (such as bearings, shear key, etc.) connecting the bridge span to an abutment will be investigated for their capacities to resist the elastic demand force.

5.1.4 Hold-down Devices at Supports. Hold-down devices at all supports or hinges in continuous structures will be investigated when the vertical seismic forces due to seismic excitation in the longitudinal direction exceeds the dead load reaction at the supports or hinges.

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5.2 Movable Steel Bridges:

The proposed methods for determining demand/capacity ratios for selected concrete bridge components (Section 5.1) may not be applicable to the three movable steel bridges in the contract. Three- and/or two-dimensional mathematical models which consider the flexibility of substructure will be developed and dynamic analysis using the response spectra method (Section 4.1) will be performed with the aid of computer program SAP IV. Two possible configurations for each bridge (i.e., both in open and closed positions) will be considered. Elastic demand forces for critical components from the results of computer analysis will be adjusted by multiplying by the proposed adjustment factor given in Section 2.2 and will be compared with the capacity values estimated as follows:

Members - $1.7\ F_s$

Connections - $1.3\ F_s$

The term F_s is the allowable working stresses in accordance with AISC Manual of Steel Construction, Eighth Edition.

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6. DEVELOPMENT OF SEISMIC RETROFITTING

Retrofitting is intended to prevent a collapse and provide for some restricted use immediately following an earthquake. The estimated ratios of demand/capacity for selected bridge components discussed in Section 5 will be used as guidelines to determine whether the bridge in question is acceptable or needs retrofitting. However, an investigation will be conducted to assess the consequences of local component failure on the overall performance of the bridge to determine the effectiveness of strengthening. Engineering judgment will be needed to justify the necessity of retrofitting.

Once the decision is made to retrofit the structure, the modified structure will be reanalyzed if retrofitting may affect the response of the remainder of the bridge. The demand/capacity ratio will be recalculated to comply with the performance criteria.

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APPENDIX B
CALTRANS RESPONSE SPECTRA

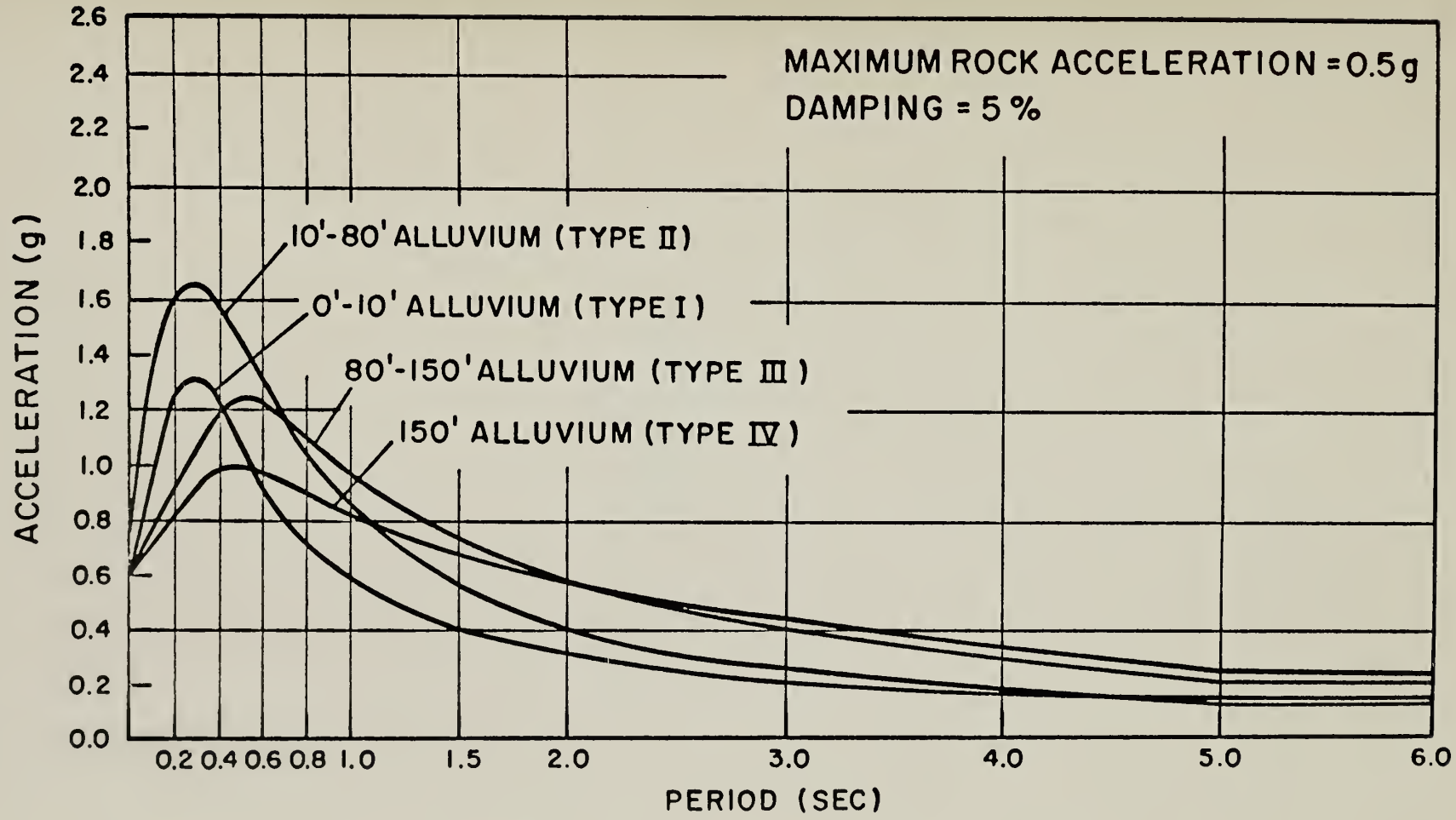

FIGURE 3a. CALTRANS RESPONSE SPECTRA
(FROM REFERENCE 3)

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APPENDIX C
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APPENDIX C

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